WATERWORKS HANDBOOK

WATERWORKS HANDBOOK

OF

DESIGN, CONSTRUCTION AND OPERATION

COMPILED BY

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PREFACE TO THIRD EDITION

In the ten years since first publication, the Waterworks Handbook has been found useful by many persons besides waterworks designers and superintendents. It has gone into a number of distant countries; it was used in large numbers by Army Engineers in the World War. In preparing the third edition the authors have tried to make it serviceable to irrigation, power, railroad, highway and other engineers, contractors and executives. However, the book is primarily a tool for waterworks men.

Since the second edition, progress has been made in the many arts upon which waterworks practice is based, and many new statistical data have accumulated. Advantage has been taken of these in the selection of new material. Parts of the Handbook have been written anew. There has been a re-arrangement of certain parts. Some passages in earlier editions which proved to be less useful have been omitted for lack of space. It is impossible to compress all the valuable material in waterworks literature and practice into one book, or to have the very latest advances recorded therein, when it reaches the purchaser. Acquisition of knowledge continues apace while proofreaders and pressmen are at work.

This compilation is necessarily a condensation of numerous experiences and excerpts from technical literature. This literature is available in several technical libraries. So far as practicable reference to sources of information is made in the bibliography at the end of each chapter. This affords opportunity for the reader interested in further research to order the full citation from the Engineering Societies Library, or some other library rich in technical journals. Large libraries are prepared to furnish photostatic reproductions of any printed page for a small charge.

The references to the bibliographies are small bold-face numbers placed slightly above the line and adjacent to the subject matter. For example, the figure 6, in the last line of page 373 refers to ref. 6 in the bibliography on page 380.

The bibliographies and the copious references throughout the text are more complete than in previous editions. The index has been improved and the self-indexing features of the arrangement and headings have been retained.

The authors express their thanks to many friends for their numerous and helpful suggestions and for data which have been utilized in the Handbook. Credits have been given in the text so far as feasible and omission of a long list of names here is not to be construed as a lack of gratitude.

THE AUTHORS.

November, 1923

PREFACE TO FIRST EDITION

This book gives a usable compilation of information, old and new, for the waterworks engineer and superintendent, the designer, constructor, operator and inspector. The materials have been accumulated by the compilers in the course of their practice in various branches of waterworks engineering. The user is assumed to have some familiarity with mathematics, hydraulics, the natural sciences and waterworks construction, operation and maintenance, and to possess ordinary mathematical tables. For some cases, instead of rules or formulas, data have been given, from which the engineer can make his own determinations, exercising his judgment in accordance with local conditions and other data which he may possess. Mention of materials, apparatus, equipment, methods, formulas, persons or business concerns, does not necessarily imply approval by the compilers. Acknowledgment of large indebtedness to many persons is here made, especially to John H. Gregory, and Thomas C. Atwood, who read the proofs and made numerous helpful suggestions; credit to many others is given throughout the text.

For convenience of reference the book has been made partially self-indexing by grouping most of the contents under the following topics, naturally arranged: Sources of Supply; Collection and Works Therefor; Transportation by Aqueducts, Pipes, and Other Conduits; Distribution, Hydrostatics and Hydraulics; Materials Much Used in Waterworks; and Treatment by Filtration, Aeration, Chemicals and Other Means. Each topic is divided on natural and obvious lines and its divisions arranged sequentially. Material not readily classified under the above topics has been assembled under the heading of Miscellany on p. 635 *et seq.*

Specifications (*e.g.*, for water towers) have been stripped of obvious and usual non-technical matter and minor words and the substance packed into little space. The user will, of course, amplify and arrange in proper form when preparing a contract. Much information which will be useful in preparing specifications has not been so labelled, but has been put into natural places in the text.

THE AUTHORS.

NEW YORK,
May, 1916.

CONTENTS

PAGE

PREFACE TO THIRD EDITION v

PREFACE TO FIRST EDITION vii

ABBREVIATIONS FOR REFERENCES xi

ABBREVIATIONS AND SYMBOLS xiii

PART I. SOURCES OF WATER SUPPLY

1. Rainfall or Precipitation 1
2. Evaporation 26
3. Run-off and Stream-flow 38
4. Ground Water 71

PART II. COLLECTION OF WATER

5. Intakes 81
6. Watershed Development by Reservoirs 85
7. Masonry Dams 116
8. Rock-fill Dams 189
9. Earth Dams or Dikes 193
10. Wells 222
11. Infiltration Galleries 260
12. Springs 266

PART III. TRANSPORTATION AND DELIVERY OF WATER

13. Open Channels 269
14. Masonry Aqueducts 280
15. Plate Metal Pipes (Steel and Iron) . . . 301
16. Wood-Stave Pipes 358
17. Concrete and Reinforced Concrete Pipes . 373

PART IV. DISTRIBUTION OF WATER

18. Cast-iron Pipe and Fittings 381
19. Distribution Systems 398
20. Valves, Sluice Gates, and Hydrants 436
21. Service Meters 459
22. Pumps, Pumping Stations and Equipment 469
23. Reserve Storage in Distribution Reservoirs, Standpipes and Tanks . . . 533
24. Water Consumption 571

PART V. CHARACTER AND TREATMENT OF WATER

25. Character of Water 583
26. Protection of Water Supply Sources 625
27. Storage of Water and Improvement of Water 633

PAGE
28. Subsidence 640
29. Aeration 645
30. Coagulation and Disinfection 650
31. Water Softening 680
32. Miscellaneous Purification Processes 695
33. Filtration 702

PART VI. HYDRAULICS AND MATERIALS

34. Hydraulic Computations 747
35. Masonry and Puddle 793
36. Metals 805
37. Capacity and Conversion Tables 815
38. Miscellany 825

INDEX 839

ABBREVIATIONS FOR REFERENCES

C. & M. E.	Chemical and Metallurgical Engineering (New York).
Eng. Contr., E. C.	Engineering-Contracting (Chicago).
E. N.	Engineering News (N. Y.).
E. N. R.	Engineering News-Record, New York.
E. R.	Engineering Record (N. Y.).
I. & E. C.	Industrial and Engineering Chemistry.
Jl. Assn. Engg. Socs., J. A. E. S.	Journal of Association of Engineering Societies (Boston).
J. N. E. W. W. A.	Journal of New England Water Works Association (Boston).
McGraw Hill	McGraw-Hill Book Company, Inc., New York.
N. E. L. A.	National Electric Light Association.
Proc. A. W. W. Assn.	Proceedings of American Water Works Association (Troy).
J. A. W. W. A.	Journal of American Water Works Association, N. Y.
Proc. Mun. Engrs.	Proceedings of Municipal Engineers of New York.
Proc. Inst. C. E.	Minutes of the proceedings of the Institution of Civil Engineers (London).
T. A. S. C. E., Trans. Am. Soc. C. E.	Transactions of American Society of Civil Engineers (New York).
T. A. S. M. E.	Transactions, American Society of Mechanical Engineers.
T. A. S. T. M.	Transactions, American Society of Testing Materials.
U. S. G. S., U. S. Geol. Survey	United States Geological Survey (Washington).
Wiley	John Wiley & Sons, Inc., New York.

ABBREVIATIONS AND SYMBOLS

(Other meanings given in text, wherever used.)

a. area or acre.
a.c. alternating current.
a.-ft. acre-foot.
av. or aver. average.
b. breadth.
B.t.u. British thermal units.
B. hp. brake horse power.
C. Centigrade.
c. coefficient in numerous formulas (also C, c^1, c_1, etc.)
cu. ft. cubic foot.
c.f.s. cubic feet per second.
c.g. center of gravity.
c.i. cast iron.
c.l. center line, or axis.
cu. yd. cubic yard.
d. day.
d.c. direct current.
diam. diameter.
deg. degree.
E. modulus of elasticity.
e. basis of natural system of logarithms = 2.71828.
F. Fahrenheit.
f. stress in extreme fibre of beam; friction loss in pipes, etc.
f.p.s. feet per second.
ft. foot, feet (United States).
g. gravity, gallon, gram.
gal. gallon (United States).
g.p. cap. gallons per capita.
g.p.d. gallons per day (24 hours).
g.p.g. grains per gallon.
g.p.m. gallons per minute.
gr. grains.
g. or grm. grams.
h. head.
hp. horsepower.
ht. height.
hr. hour.
hyd. gr. hydraulic gradient.
I. moment of inertia.
i. inclination, slope-gradient friction-head.
in. inch (United States).
L. liter.
l. length.
kw. kilowatt.
lb. pound (avoirdupois).
lb. sq. in. pounds per square inch.
lin. linear.
M. moment of forces.
m. meter.
max. maximum.
mi. mile.
min. minimum; minute of time.
mg. million gallons and milligrams.
mgad. million gallons per acre daily.
mgd. million gallons daily.
mo. month.
n. rugosity factor, or coefficient of roughness.
π ratio of circumference of circle to diameter = $3\frac{1}{7}$, 3.1416, 3.14159265359.
p. perimeter, pressure, wetted perimeter.
P.C. point of curvature.
pH. hydrogen ion concentration.
P.I. point of intersection.
p.p.m. parts per million.
P.T. point of tangency.
Q, q. quantity or discharge.
r. hydraulic mean radius, radius, revolutions.
r.p.m. revolutions per minute.
s. second of time, slope.
sp. g. specific gravity.
sq. ft. square foot.

sq. in. square inch.
sq. mi. square mile.
T. ton (2000 pounds); tensile stress.
t. thickness; time.
T. sq. ft. tons per square foot.
v. velocity, mean velocity.
y. yard; distance from neutral axis of beam to extreme fibre.
yr. year.
′ foot, minute, prime.
″ inch, second.
° degree.

"IF WE only take into consideration the abundant supply of water to the public, for baths, ponds, canals, household purposes, gardens, places in the suburbs, and villas; and then reflect upon the distances that are traversed, the arches that have been constructed, the mountains that have been pierced, the valleys that have been filled up, we must of necessity admit that there is nothing to be found more worthy of our admiration throughout the whole universe." (*Pliny*)

WATERWORKS HANDBOOK

PART I
SOURCES OF WATER SUPPLY

CHAPTER 1

RAINFALL OR PRECIPITATION

Value of Rainfall Data. Rainfall statistics have been systematically collected by U. S. Weather Bureau for nearly 55 years, and cover practically the whole country. Although the engineer is interested primarily in run-off data, their meagerness and brevity in comparison with those on rainfall force him often to fall back on rainfall statistics for approximating stream flow, and for studying prolonged dry periods. Similarly, in studying spillway and culvert capacity, use can be made of rainfall data to indicate the greatest flow to be expected.

RAIN GAGES*

U. S. Weather Bureau standard gage consists of a galvanized-iron can, 8 in. in diam., and 20 in. high, provided with a funnel-shaped top, tightly fitting, which has a sharply beveled brass rim, accurately circular, so as to catch only the rainfall on a definite area. The bevel is on the outside, so that water striking it does not splash into the can. The funnel has an opening (made small to prevent evaporation) into an inner vertical brass cylinder of one-tenth the horizontal area of the rim of the funnel. The wetted length of a slender measuring rod graduated to tenths of an inch, inserted in this brass cylinder, measures ten times the actual fall, and determines precipitation to hundredths. The displacement of the measuring stick is ignored, as it is small. The brass cylinder will hold 2 in. of rainfall before it overflows into the annular space between the cylinders, called the overflow. To measure this overflow, pour into a convenient vessel, and then back into the cylinder.

Automatic rain gages† should be restricted to locations where they can be given frequent attention, as they are liable to derangement. Severe winter weather is decidedly against delicate long-term apparatus.

Errors. Comparison of rain gages at Ithaca, N. Y., 1908–1910 by State Conservation Commission,[1] showed that variations of registrations by "old

* See also "The Measurement of Rainfall and Snow," by R E Horton, *J. N. E. W. W. A.*, Vol. 33, 1919, p 14; and S P Fergusson in *Monthly Weather Review*, February 1922, p. 82.

† Manufacturers include J. P Friez, Baltimore, Md ; Queen-Gray Co, Philadelphia; Jules Richard, Paris; International Instrument Co , Cambridge, Mass; and Draper Mfg. Co., N. Y.

types" were not great. The U. S. Weather Bureau Standard gage was on the ground, in a row with three others; its value was assumed as 1.000. The elevated gages were 20 ft. away.

Make	Value on ground	Value 10 ft. above ground
Smithsonian	1.000	1.041
Fuertes	1.016	1.046
Dewitt conical	0.971	0.968

Meyer[2a] presents the following observed ratios to show effect of wind at higher levels, unity value being on ground: 43 ft. high, catch is 0.75; 85 ft. high, catch is 0.64; and 194 ft. high, catch is 0.58. G. F. Swain[3] reports value of 0.94 for 1 ft. above ground. Some records published in *Monthly Weather Review* are from gages situated atop high buildings.

Cleveland Abbe says that no error of more than 1 per cent. systematically attaches to gages of ordinary forms and of diameters between 4 and 44 in. Old rain gages, placed 8 ft. above ground, gave uniformly low results, on account of: (1) loss by evaporation; (2) method of measuring snowfall; (3) height above ground.

Location of gages is important. Place them in open spaces free from obstructions such as steep slopes, trees, buildings, fences, and at least 100 ft. distant. On the Catskill works, a Weather Bureau gage too near a house gave results at variance by 5 in. in the 1911 record (total about 45 in.) from those of a similar gage favorably located on an open lawn several hundred feet distant. This whole difference is considered chargeable to location. In a large city, gages should be placed on flat roofs well away from the edges.

RAINFALL DATA

Sources. The climatological data published by U. S. Weather Bureau in *Monthly Weather Review* have been summarized to, and including, 1920, in "Summaries of Climatological Data by Sections."

Detailed tables of monthly totals for New England have been compiled by X. H. Goodnough in *J. N. E. W. W. A.*, September 1915, and September 1921. Annual reports of many municipal departments and state commissions contain rainfall tables extending over terms of years. A notable report is *Bull.* 5, State of Cal., Dept. Pub. Works, Div. of Eng. and Irrigation, "Flow in California Streams," 1920, wherein summaries are given. Meyers in "Elements of Hydrology" (Wiley, 1917), and Mead in "Hydrology" (McGraw-Hill Book Company, Inc., 1919), summarize various records. Little faith should be put in many American records previous to 1880.

Form of Data. Published records generally give for average the arithmetic mean; some engineers[4] prefer the median, defined on p. 63. U. S. Weather Bureau uses the calendar year in presenting totals and averages. C. E. Grunsky[5a] points out the absurdity of reckoning by the calendar year when utilizing rainfall records for run-off comparisons. Statistics are customarily recorded to hundredth of inch, but this degree of precision is hardly warranted in practice; the added labor yields no gain in accuracy.

The "water year," also termed "seasonal" and "climatic," is a 12-month interval established by hydrologists to take account of the lag of run-off behind precipitation. The "water year" in most regions does not coincide with the calendar year, nor is it necessarily the same time interval in successive years. Records of U. S. Geological Survey in Chap. 3 adopt the uniform time interval, October 1 to Sept. 30. In northern latitudes, A. F. Meyer[2b] uses, for rainfall, the 12 months beginning Nov. 1, and for corresponding run-off the year beginning Mar. 1. Wherever the records are compiled from data of U. S. Weather Bureau, tables in this book follow the calendar year.

Estimating Precipitation. If a mean is to indicate the character of the phenomena from which it is derived, it must be computed from sufficiently representative data. Table 2 illustrates the fallacy of basing conclusions on data from short periods, and Table 1 indicates the difference in neighboring stations in recording the dry years, 1880 and 1883

One short-term record may be used if there exists for an adjacent or a comparable region, a long-term record embracing the years of the short term. From the long-term record, compute the annual mean for those years corresponding to the short-term record and find its ratio, R, to the long-term mean. Then R times the annual mean of the short-term record gives a product analogous to the mean of the long-term record. Similarly a minimum (or a maximum) year would be calculated from the mean thus found by applying the ratio found from the long-term record of Minimum Mean (or Maximum Mean). Table 3 gives some values for these ratios

Table 1. Variation in Dry-year Records for Stations within an Area 30 by 80 Miles (L M Hastings[6])

Location	40 year average (1874 1913)	Dry year records	
		1880	1883
Manchester, N H	38.3	27.3	31.5
Concord, N H	38.6	30.5	31.4
Lowell, Mass	42.0	35.3	39.8
Waltham, Mass	43.4	31.7	29.3
Cambridge, Mass	43.5	35.2	32.6
Framingham (Sudbury River) Mass	44.3	37.9	32.0
New Bedford, Mass	46.6	40.1	43.5
Providence, R I	47.2	41.3	39.5

Method of Combining Records, to Get Mean (1) Take the mean of all records, including broken ones. Averaging by months allows this, whereas a yearly average could be based only on complete annual records.* (2) Combine the stations into groups equally distributed. Determine the mean for each group and take the average of these means as the average for the whole area. This has the advantage over No. 1 that too much weight is not given to any particular locality. (3) Take the mean of the inside stations, and the mean of the outside stations, of the area considered, averaging these two means. This method is good when there is a greater number of outside stations, as it gives more weight to the inside stations. (4) Average monthly and yearly rainfalls on Catskill watersheds are determined by weighting each station in

* Horton[20] recommends using the mean of three surrounding stations for a missing month.

proportion to the percentage which the area represented by its gage (generally considered a circle with center at the gage) is to the total area of watershed. Weight × rainfall, summed up and divided by 100 (weight of total watershed) = average rainfall. This method gives results that agree well with (1) when the rainfall over whole watershed has been fairly uniform. Differences of several tenths of an inch in the two methods are not uncommon, but are, as a rule, compensating. For heavy rainfall, the weighted method approaches more nearly the truth.*

(5) *Isohyetal Map.* Procure a scale map of the locality to be studied. Locate the positions of all rain gages. At each gage mark its average rainfall. Averages should, if possible, be taken for same number of years and for same period. Treat the problem similarly to plotting contours on a topographical map, drawing "contours" through points of equal rainfall, using straight-line interpolations and extrapolations, where necessary. Good, long-term records of large number of gages well distributed over the area should be available.

Table 2. Average and Extreme Departures of Mean Rainfall by Short Periods from Mean Determined by Very Long Periods, Expressed in Per Cent.

Station	Long period, years	5 yrs.	10 yrs.	20 yrs.	40 yrs.
Padua[7]	176	9.6	8.4	2.5	2.4
Klangenfust[7]	88	9.5	8.1	2.6	2.6
Milan[7]	100	7.0	5.9	2.7	2.0
Croton	53	−12.4 +13.6	− 6.4 +12.2	− 4.3 + 3.1	−1.2 +0.7
Sudbury	46	−13.5 + 9.3	−13.0 + 3.0	− 6.7 + 2.4	0.0
† San Francisco	71	−25.7 +20.8	−16.7 +12.8	− 8.4 + 6.6	−2.6 +5.7
Denver	49	−21.0 +11.9	−11.9 +12.6	− 7.7 + 2.1	0.0
St. Paul	85	−19.5 +27.7	−11.6 +17.5	−10.5 +12.1	−1.1 +5.1
New Orleans	75	−16.1 +19.5	−13.5 +14.7	− 6.9 + 6.9	1.2
Cincinnati	65	−24.9 +31.5	−17.0 +19.5	−12.9 +10.5	−2.5 +5.1

† Years 1839–1860, 1873–1920.

Factors Affecting Estimates of Rainfall. Usual pathway of storms across the region under consideration as well as temperature are important elements in determining the precipitation probable at a given point. Owing to the local conditions, especially adjacent buildings, Weather Bureau records in large cities are particularly unreliable. Meteorologists agree that the amount of precipitation on a given area is "directly proportional to its altitude and inversely proportional to its distance from the ocean or other large body of water over which the prevailing storm winds pass." (Kuichling.) Studies by H. N. Savage, San Diego, Cal. show the influence of elevation on seasonal rainfall in southern California to be an increase of 6 in. for each 1000 ft. elevation. Hazen applied a reduction of 0.64 in. run-off per 100 ft. elevation in his

* See Thiessen in *Monthly Weather Review*, July 1911, p. 1082.

Report on Water Resources of New Jersey (1922).* **Seaward slopes of mountain ranges which intercept moisture-laden winds receive copious precipitation, while landward slopes and interior regions receive less, often much less. (The west coast of South America is an exception.) In some localities, precipitation on tops of mountains and high, wooded hills is greater than in adjacent lowlands. Adjacent stations often produce widely varying records: *e.g.* Albany and Troy, 7 mi. apart; Newark and New York, 10 mi. apart. Records for the same storm at stations but a few miles apart often vary. Moore[22] cites a difference of monthly results averaging 5 per cent. in Berlin from gages, but 1500 ft. apart. Greatest discrepancies occur in the months of thunderstorms.**

Variation in annual rainfall has been investigated by Binnie,[7] Birkinbine[8] Hazen, Wells, and others, as a basis for rainfall predictions and other hydraulic studies. Hazen's[9] studies of normal coefficients of variation indicate that the Atlantic Coast, Great Lakes, and Puget Sound regions are more likely to get normal rainfall. Wells[10] concluded from a study of 226 long-time records, that generally the longer the record, the lower will be the value of ratio, minimum year to mean. Data from his lengthy table are given in Table 3.

Table 3. Rainfall Factors for United States

J. P. Wells[10]

Stations	Years	Mean inches	Ratio to Mean Max	Ratio to Mean Min.	Stations	Years	Mean inches	Ratio to Mean Max.	Ratio to Mean Min.
Me. Eastport	36	42.9	1.5	0.53	Ill. Peoria	53	34.8	1.5	0.70
Vt. Burlington	64	32.7	1.5	0.64	Wis. Madison	41	31.2	1.7	0.43
Mass. Boston	91	44.7	1.6	0.60	Minn. Duluth	38	29.8	1.5	0.58
Mass. Sudbury	46	44.6	1.3	0.74	Minn. St. Paul	72	27.8	1.8	0.53
Conn. Hartford	47	44.3	1.3	0.75	Iowa, Davenport	37	32.6	1.4	0.50
R. I. Providence	77	45.3	1.4	0.67	Mo. St. Louis	72	40.1	1.7	0.57
N. Y. Ithaca	48	33 0	1.4	0.66	Kan. Dodge City	42	19.6	1.7	0.39
N. Y. Croton	53	48.3	1.3	0.75	Neb. Omaha	41	30.5	1.6	0.58
Penn. Erie	35	38.2	1.3	0.70	S. Dak., Yankton	32	26.0	1.7	0.55
Penn. Philadelphia	38	40 9	1.3	0.75	N. Dak., Bismarck	34	17.5	1.8	0.53
N. J. Newark	65	47.8	1.4	0.65	La. New Orleans	63	55.6	1.5	0.55
Md. Baltimore	39	43.1	1.5	0.73	Ark. Hope	41	52.7	1.3	0.55
D. C. Washington	72	40.8	1.5	0.46	Texas, El Paso	40	9.7	2.4	0.25
W. Va. Morgantown	32	44.9	1.5	0.62	Mont. Miles City	31	12.8	1.8	0.71
Va. Richmond	36	43.0	1.7	0.64	Wyo. Cheyenne	37	13.8	1.6	0.36
N. C. Wilmington	38	50.9	1.6	0.79	Idaho, Lewiston	28	14.7	1.5	0.66
S. C. Charleston	115	48 6	1.6	0.48	Okla. Ft. Sill	35	30.8	1.6	0.52
Ga. Augusta	40	46.9	1.2	0.40	Wash. Spokane	28	17.9	1.5	0.67
Fla. Tampa	40	51.5	1.8	0.64	Ore. Astoria	37	37.1	1.7	0.51
Ala. Mobile	37	61.8	1.5	0.63	Ore. Portland	50	42.4	1.6	0.72
Miss. Vicksburg	54	59.1	1.6	0.71	Cal. Sacramento	60	19.4	1.8	0.45
Tenn. Chattanooga	30	50.1	1.4	0.65	Cal. San Diego	60	9.5	2.9	0.32
Ky. Louisville	36	44.0	1.4	0.66	Nev. Beowaine	36	6.5	2.3	0.32
O. Toledo	47	32.6	1.4	0.65	Utah, Ogden	37	14.7	1.6	0.44
Mich. Detroit	38	32.1	1.5	0.65	Col. Denver	37	14.0	1.4	0.60
Mich. Marquette	37	32.5	1.3	0.78	Ariz. Tucson	40	11.7	2.1	0.45
Ind. Indianapolis	37	41.1	1.4	0.73	N. M. Santa Fé	50	14.7	1.7	0.53

Table 4. Relation of Wettest and Driest Years[21]

Mean rainfall, in inches	Wettest year, percentage of mean	Dryest year, percentage of mean	Range ratio, percentage
5–30	178	55	123
30–40	154	54	100
40–50	143	64	79
50–60	142	70	72

* The effect on rainfall may be less.

Rendering Records Comparable. Great differences in records exist; the following is a method of rendering them comparable suggested by Thaddeus Merriman: Any monthly rainfall that exceeds twice the monthly mean is excessive or unusual, and should be eliminated, as follows: For any month in which the rainfall exceeded twice the monthly mean, use the monthly mean, unless the rainfall for either the preceding or following month was less than half its monthly mean, in which case deduct only the excess of the 1 month over the deficiency of the 2 months. The value of the yearly rainfall so determined may be called the "mean annual dependable" rainfall.

Where rainfall stations are widely separated, as in the Northwestern States, it is the experience of the U. S. Geological Survey that calculations are facilitated by expressing each year's rainfall for any station as a percentage of the mean annual of that station for many years. Percentages of several stations averaged together for any year convey an idea of the comparative dryness in the drainage basin, without introducing an error due to local conditions at the stations. Results thus obtained are said to be satisfactory, making averages for large areas much more accurate.*

In comparing rainfalls at two localities of short records, both should be reduced to a common plane for comparison with long-time records. Data for Northeastern United States show that the proposition that the rainfall varies from the mean at the same rate for all stations similarly located is true 72 per cent. of the time. The smaller the area, the less is the error.

Frequency Curves. Rainfall is irregular, wet and dry years usually occurring in cycles of several years' duration. Studies of cycles, frequency of occurrence, and seasonal variation of rainfall can be aided by the use of frequency curves as outlined by Foster in "Theoretical Frequency Curves and their Applications to Engineering Problems."† Horton studied Padua, Italy, records and concluded that the evidence of periodicity was too slight to be significant.‡

Snowfall Expressed in Inches of Rainfall. The ratio of snow to rain varies greatly with the character of the snowfall. U. S. Weather Bureau divides snowfall by 10 to get equivalent rain. The one-tenth rule gives values 10 to 30 per cent large for light snow at low temperatures. Other authorities recommend divisors varying from 7 to 12; the last was used for Potsdam, Prussia, and Catskill Mountains.§ Meyer[2c] estimates that all precipitation is snowfall when monthly mean temperature is below 20°F.

RAINFALL RECORDS

Explanation of Rainfall Tables. The following tables were compiled mainly from U. S. Weather Bureau records, dropping the second decimal place. Elevations refer to height of gage above sea-level. "Totals" in the second column, and "Extent of Record" below each table, enable an investigator in any year to bring the record quickly down to date by adding records of subsequent years. Records are inclusive of first and last dates

* This method is also endorsed by C. E. Grunsky, "Rainfall and Run-off Studies," *T. A. S. C. E.* Vol. 85, 1922, p. 69.

† *T. A. S. C. E.*, Vol. 87, 1924, p. 142.

‡ See *Monthly Weather Review*, October, 1923, p. 515. See also Saville on "Rainfall Data Interpreted by Laws of Probability," in *E. N.*, Dec. 28, 1916, p. 1208, and Tolley in *Monthly Weather Review*, November, 1916, p. 634.

§ See "Snowfall of United States," by R. D. Ward, in *Scientific Monthly*, November, 1919, p. 397.

given. Rainfall is expressed in inches, and includes snowfall expressed in inches of rain. Figures for the lowest 5 and highest 7 consecutive months are

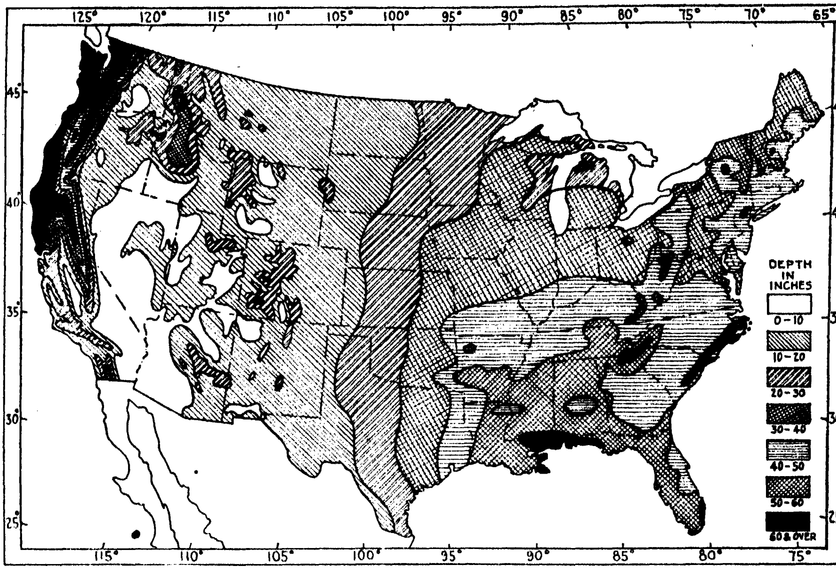


FIG. 1.—Mean annual rainfall of United States.
(U. S. Weather Bureau, 1917.)

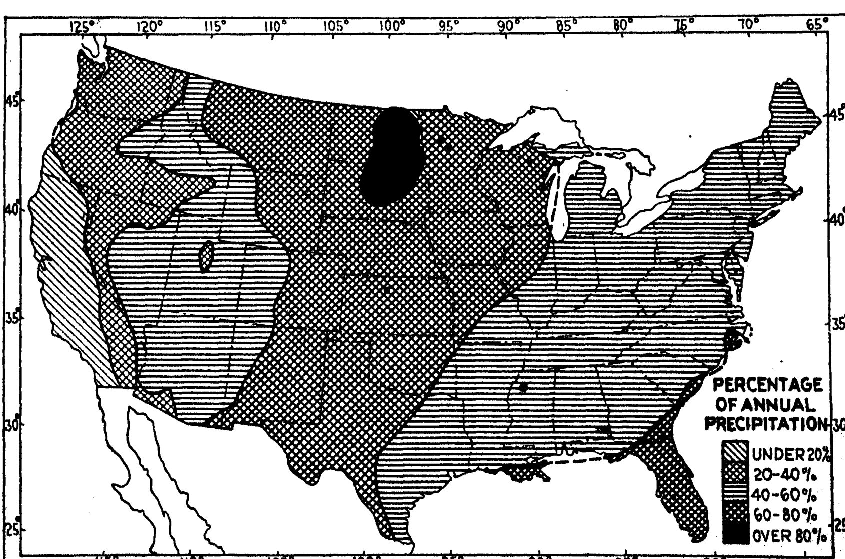


FIG. 2.—Percentage of annual precipitation occurring between April 1 and September 30.
(U. S. Weather Bureau, 1917.)

the total rainfalls during these periods; they are useful for studies in watershed development. Temperature is in degrees Fahrenheit. Places are arranged geographically.

Table 5. Selected Long-term Rain and Temperature Records Giving Monthly and Yearly Totals, Averages, Minima and Maxima, also Extreme Dry and Wet Seasons

Minimum Minimorum and Maximum Maximorum, also Lowest and Highest Average, Are Underscored for Each Place

Place	QUEBEC, Canada., Elev. 296 ft.						
	Rainfall, inches						Mean temp., °F. (24 years) (1897–1920)
			Minimum		Maximum		
Time	Totals 47 yrs.	Average	Year	Amount	Year	Amount	
Jan......	171.7	3.7	1918	0.3	1882	6.6	10
Feb......	151.4	3.2	1878	1.0	1908	6.2	11
Mar......	155.6	3.3	1915	0.4	1877	6.2	23
Apr......	106.0	2.3	1883, 1886	0.7	1880	6.6	35
May.....	153.5	3.3	1887	0.3	1874, 1919	6.9	51
June.....	190.5	4.1	1881	1.3	1901	9.2	60
July......	196.9	4.2	1877	0.5	1919	8.1	67
Aug......	182.9	3.9	1905	1.4	1912	9.6	63
Sept	183.6	3.9	1874, 1903	1.1	1918	9.4	55
Oct......	162.5	3.5	1895	0.9	1896	7.0	44
Nov......	156.4	3.3	1876	0.9	1878	7.1	30
Dec......	159.7	3.4	1892	1.1	1878	6.8	17
Year...	1962.2	41.8	1905	32.3	1879	52.4	40

Lowest 5 consec. months, Oct., 1892–Feb., 1893, 8.5.
Highest 7 consec. months, May–Nov., 1918, 38.9.
Extent of record 47 yrs, 1874–1920. Lowest Jan. temperature, -31°.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	TORONTO, Ontario. Elev. 350 ft. until 1908; 377 ft. since						
	Rainfall, inches						Mean temp., °F. (30 yrs.)
			Minimum		Maximum		
Time	Totals 47 yrs.	Average	Year	Amount	Year	Amount	
Jan......	129.2	2.8	1920	0.6	1886	5.5	22
Feb......	111.6	2.4	1877	0.3	1900	5.0	22
Mar......	113.2	2.4	1905	0.5	1876	5.7	29
Apr......	107.4	2.3	1881	0.1	1909	5.4	41
May.....	131.2	2.8	1920	0.4	1894	9.1	53
June.....	125.3	2.7	1899	0.6	1892	5.8	63
July......	133.8	2.8	1916	0.4	1878, 1883	5.6	68
Aug......	131.4	2.8	1876	0.0	1915	8.1	67
Sept.....	125.5	2.7	1877, 1897, 1903	0.4	1878	7.7	59
Oct......	119.4	2.5	1901	0.5	1898	5.4	47
Nov......	123.2	2.6	1904	0.1	1877	5.6	36
Dec......	115.4	2.4	1877	0.5	1889	4.9	26
Year...	1466.6	31.2	1874	24.3	1878	48.5*	44

Lowest 5 consec. months, Feb.–June, 1895, 6.0.
Highest 7 consec. months, June–Dec., 1878, 33.6.
Extent of record, 47 yrs., 1874–1920.
Lowest Jan. temperature, -23°.

* Wells[10] gives 50.8 for 70 years.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	BOSTON, Mass Elev 125 ft						
	Rainfall, inches						Mean temp., °F (50 yrs)
			Minimum		Maximum		
Time	Totals, 103 yrs	Average	Year	Amount	Year	Amount	
Jan	387 8	3 8	1849	0 4	1836	8 8	27
Feb	378 3	3 7	1901	0 7	1869	10 0	28
Mar.	424 5	4 1	1915	1 0	1864	11 8	35
Apr	401 3	3 9	1844	0 2	1857	10 8	45
May	371 7	3 6	1826	0 2	1868	10 4	57
June	321 2	3 1	{1853, 1912}	0 3	1858	8 1	66
July.	366 2	3 6	1849	0 8	1863	12 4	72
Aug	422 4	4 1	1883	0 3	1826	12 1	69
Sept	354 4	3 4	{1884, 1914}	0 2	1868	12 0	63
Oct.	368 6	3 6	1897	0 4	1890	8 8	53
Nov.	411 4	4 0	1917	0 5	1840	11 6	42
Dec	388 3	3 8	1828	0 3	1869	9 0	32
Year	4594 6	44 6	1822	27 2	1863	67 7	49

Lowest 5 consec months, Oct, 1818–Feb, 1819, 7 9
Highest 7 consec months, Oct, 1869 Apr, 1870, 47 9
Extent of record, 103 yrs, 1818–1920 Lowest Jan temperature, –13° (50 yrs).
In 47 days, Sept and Oct, 1914, but 0 21 in fell (E N, Apr. 8, 1915) Records for 1856–1878, and for the late 80's are not reliable according to Goodnough, *J. N. E. W W. A.*, Sept. 1915

Selected Long-term Rain and Temperature Records —(*Continued*)*

Place	SUDBURY WATERSHED, Mass Elev 200 ft						
	Rainfall, inches						Mean temp., °F (21 yrs) (1898–1918)
			Minimum		Maximum		
Time	Totals, 46 yrs	Average	Year	Amount	Year	Amount	
Jan	184 9	4 0	1916	1 5	1891	7 0	28
Feb	190 9	4 2	1877	0 7	1900	9 1	25
Mar.	199 2	4 3	1915	0 1	1877	8 4	38
Apr	164 7	3 6	1892	0 8	1904	8 9	47
May	152 3	3 3	1903	0 9	1901	7 2	57
June	144 9	3 2	1812	0 5	1903	9 2	66
July.	167 4	3 6	1917	1 1	1876	9 0	72
Aug	175 3	3 8	1883	0 7	1898	8 2	69
Sept	157 3	3 4	{1877, 1914}	0 3	1907	8 8	62
Oct	168 8	3 7	1897	0 5	1895	10 7	52
Nov.	172 5	3 8	1908	1 0	1888	7 2	40
Dec	174 3	3 8	{1875, 1877}	0 9	1901	9 7	29
Year	2052 5	44 6	1883	32 8	1878	57 9	49

Lowest 5 consec. months, Apr.–Aug., 1899, 10.4.
Highest 7 consec. months, Sept, 1899–Mar., 1891, 41 7
Extent of record, 46 yrs., 1875–1920.
Lowest Jan. temp., –24° (13 yrs).

* See also "Rainfall in New England," by X H Goodnough in *J N E W W A*, September 1915 and 1921 for tabulation of 250 New England records Also p 45

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place: WACHUSETT WATERSHED, Mass. Elev. 400 ft.	Rainfall, inches						Mean temp., °F. (21 yrs.) 1898–1918
Time	Totals, 24 yrs.	Average	Minimum Year	Minimum Amount	Maximum Year	Maximum Amount	
Jan.	86.2	3.6	1916	1.6	1898	6.6	25
Feb.	93.4	3.9	1901	1.1	1900	8.7	23
Mar.	96.2	4.1	1915	0.1	1899	6.8	34
Apr.	90.2	3.8	1915, 1917	1.8	1901	9.6	45
May	82.3	3.4	1905	0.8	1901	7.0	57
June	90.7	3.8	1912	0.5	1903	10.4	65
July	96.2	4.1	1917	1.2	1897, 1915	8.6	71
Aug.	96.2	4.1	1907	1.3	1898	10.6	68
Sept.	92.2	3.8	1914	0.2	1907	9.5	61
Oct.	77.5	3.2	1920	0.6	1898	7.2	51
Nov.	83.3	3.5	1902	0.9	1897	7.6	39
Dec.	97.4	4.0	1899	2.0	1901	9.4	28
Year	1087.7	45.3	1917	37.3	1898	57.9	47

Lowest 5 consec. months, Feb.–June, 1915, 10.1.
Highest 7 consec. months, Aug., 1898–Mar., 1899, 39.8.
Extent of record, 24 yrs., 1897–1920.
Lowest Jan. temp., −16° (13 yrs.).
See also p. 46.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place: ESOPUS CREEK WATERSHED, Catskill Mts., N. Y. (6 to 11 stations). Elev. 600–1200	Rainfall, inches						Mean temp., °F. (9 yrs.) 1906–1914
Time	Totals, 15 yrs., 3 months	Average	Minimum Year	Minimum Amount	Maximum Year	Maximum Amount	
Jan.	51.6	3.4	1916	1.6	1910	7.4	27
Feb.	51.1	3.4	1907	1.7	1909	6.9	23
Mar.	55.0	3.7	1915	0.2	1913	7.7	35
Apr.	61.5	4.1	1907, 1915	2.2	1910	9.6	47
May	59.2	4.0	1911	1.1	1908	9.1	58
June	61.0	4.1	1913	1.1	1917	6.5	66
July	66.3	4.4	1913	1.7	1915	8.4	72
Aug.	65.5	4.4	1907	1.1	1915	8.9	69
Sept.	73.2	4.9	1914	0.6	1907	11.2	62
Oct.	65.7	4.1	1910	1.1	1911, 1915	7.5	52
Nov.	56.1	3.5	1908	0.6	1907	6.9	40
Dec.	58.8	3.7	1910	2.2	1915	5.8	29
Year	710.9	47.4	1914	39.7	1920	54.3	48

Lowest 5 consec. months, Oct., 1910–Feb., 1911, 11.1.
Highest 7 consec. months, Sept., 1907–Mar., 1908, 43.2.
Extent of record, 15 yrs., 3 months, Oct. 1, 1905–Dec., 1920, except 1905 total.
Mar., 1915, averaged 0.25 (0.08 to 0.66)—lowest rainfall in the 15 yrs. of Board of Water Supply records.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	PROVIDENCE, R. I. Elev. 160 ft.						
	Rainfall, inches						Mean temp., °F. (34 yrs.) (1887–1920)
Time	Totals, 89 years	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	356.9	4.0	1843	0.6	1891	8.1	31
Feb......	336.9	3.8	1877	0.3	1886	11.3	30
Mar......	361.5	4.1	1915	0.1	1876	9.8	36
Apr......	330.5	3.7	1844	0.7	1904	9.4	44
May.....	318.7	3.6	1903	0.6	1868	10.6	54
June.....	284.4	3.2	1832	0.3	1842	9.7	63
July......	293.1	3.3	1838, 1909	0.6	1898	10.3	69
Aug......	352.4	4.0	1854	0.3	1875	8.8	69
Sept.....	294.8	3.3	1855	0.2	1888, 1899, 1907	9.2	64
Oct......	314.2	3.5	1897	0.5	1890	9.2	55
Nov......	338.0	3.8	1917	0.3	1854	9.2	44
Dec......	346.6	4.0	1875	1.0	1901	9.4	35
Year...	3927.8	44.1	1914	29.5	1898	63.5	50

Lowest 5 consec. months, July–Nov., 1837, 7.2.
Highest 7 consec. months, Aug., 1888–Feb., 1889, 43.0.
Extent of record, 89 yrs., 1832–1920.
Lowest Jan. temp., –10° (34 yrs.).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	NEW BEDFORD, Mass. Elev. 88 ft.						
	Rainfall, inches						Mean temp., °F. (12 yrs.)
Time	Totals, 107 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	427.7	4.0	1839	0.8	1915	9.9	32
Feb......	414.6	3.9	1818	0.9	1814	8.3	31
Mar......	454.3	4.2	1915	0.1	1890	9.8	41
Apr......	418.8	3.9	1846	1.2	1841	9.3	50
May.....	422.7	3.9	1822	0.6	1868	9.4	61
June.....	334.6	3.1	1912	0.1	1862	8.1	70
July......	351.2	3.2	1909	0.7	1830	12.0	76
Aug......	441.9	4.1	1854	0.2	1826	18.7	73
Sept.....	366.3	3.4	1865	0.3	1850	12.1	66
Oct......	411.7	3.7	1918	0.5	1890, 1913	10.1	58
Nov......	443.5	4.1	1899	1.1	1897	9.7	46
Dec......	439.4	4.1	1828	0.4	1901	10.0	36
Year...	4926.8	46.0	1918	29.2	1829	65.4	52

Lowest 5 consec. months, Nov. 1917–Mar. 1918.
Highest 7 consec. months, July, 1830–Jan., 1831, 47.5.
Extent of record, 107 years, 1814–1920. See X. H. Goodnough, *E. N.*, Nov. 19, 1914, p. 1014; and reports of City Engineer.
Lowest Jan. temperature, –7° (12 yrs).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place / Time	CROTON WATERSHED, N. Y. Elev. 200–600 ft.						
	Rainfall, inches						Mean temp., °F. (1912)
	Totals, 53 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	214.7	4.0	1896	1.1	1891	9.1	25
Feb......	213.4	4.0	1901	0.8	1900	7.7	31
Mar......	220.8	4.2	1915	0.3	1877	8.1	40
Apr......	191.4	3.6	1892	1.1	1901	8.2	51
May.....	200.6	3.8	1887	0.3	1868	8.8	65
June.....	187.0	3.5	1873	0.7	1903	11.3	72
July......	244.1	4.6	1868	2.1	1897	12.5	77
Aug......	257.4	4.9	1899	0.6	1898	11.5	72
Sept.....	217.4	4.1	1914	0.3	1882	14.6	67
Oct......	210.3	4.0	1879	0.7	1913	9.6	61
Nov......	198.1	3.7	1902	0.9	1889	8.5	47
Dec......	207.3	3.9	1892	1.0	1901	8.8	39
Year...	2561.7	48.3	1917	36.3	1901	63.7	54

Lowest 5 consec. months, Dec., 1871–Apr. 1872, 10.8.
Highest 7 consec. months, Mar.–Sept., 1901, 46.8.
Extent of record, 53 yrs. 1868–1920.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place / Time	ALBANY, N. Y. Elev. 85 ft.						
	Rainfall, inches						Mean temp., °F. (95 yrs.) (1826–1920)
	Totals, 95 years	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	243.8	2.6	1860	0.1	1836	7.3	23
Feb......	231.4	2.4	{1856, 1877}	0.4	1870	5.2	24
Mar......	260.3	2.7	1915	0.1	1843	7.4	33
Apr......	257.7	2.7	1892	0.6	1857	7.0	47
May.....	322.9	3.4	1903	0.2	1833	8.5	59
June.....	366.6	3.9	1864	0.8	1862	8.7	68
July......	382.9	4.0	1849	0.7	1871	9.4	72
Aug......	361.5	3.8	1854	0.6	1871	10.6	70
Sept.....	312.6	3.3	1914	0.5	1890	8.9	63
Oct......	315.6	3.3	1882	0.3	1869	13.5	50
Nov......	276.1	2.9	1908	0.4	1830	7.3	39
Dec......	249.9	2.6	1828	0.2	1878	6.2	28
Year...	3581.3	37.6	1913	26.4	1871	56.8	48

Lowest 5 consec. months, Jan.–May, 1860, 5.1.
Highest 7 consec. months, Feb.–Aug., 1871, 45.3.
Extent of record, 95 yrs., 1826–1920.
Lowest Jan. temp., –34° (46 yrs).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place / Time	PEQUANNOCK WATERSHED, Newark, N. J. Elev. 700 ft. Rainfall, inches: Totals, 28 yrs.	Average	Minimum: Year	Minimum: Amount	Maximum: Year	Maximum: Amount	Mean temp., °F. (30 yrs.) (1891–1920)
Jan.	97 9	3.5	1916	1.3	1905	6.9	27
Feb.	106.3	3.8	1895	0.2	1896	9.3	26
Mar.	111 8	4.0	1915	0.7	1901	7.3	37
Apr.	113 8	4.1	1896	0.8	1909	8.8	47
May	109 7	3 9	1903	0.3	1908	7.8	57
June	113 6	4.1	1913	1 5	1903	11 0	65
July	135.3	4.8	1894	1.0	1897	14.2	69
Aug.	129 8	4 6	1914	1.8	1901	12.1	67
Sept.	118 8	4 2	1914	0.3	1907	12.6	62
Oct.	116 0	4.1	1909	1 0	1903	13.7	52
Nov.	95.1	3.4	1908	0.8	1907	6.4	40
Dec.	109 3	3.9	1896	0.9	1901	9.0	30
Year	1360.1	48 6	1917	36 6	1903	64.8	49

Lowest 5 consec. months, Nov., 1917–Mar., 1918, 10.3.
Highest 7 consec. months, Mar., 1901–Sept., 1901; Sept., 1907–Mar., 1908, 49.3.
Extent of record, 28 yrs, 1893–1920.
Lowest Jan. temp., –26°.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place / Time	PHILADELPHIA, Pa. Elev. 117 ft. Rainfall, inches: Totals, 101 yrs	Average	Minimum: Year	Minimum: Amount	Maximum: Year	Maximum: Amount	Mean temp., °F. (43 yrs.) (1878–1920)
Jan.	331 5	3.3	1821	0.5	1841	7.8	32
Feb.	318.2	3 2	1864	0.6	1896	6.9	33
Mar.	351.6	3 5	1910	0.4	1912	9.1	40
Apr.	340.1	3.4	1847, 1881	0.6	1874	9.8	51
May	373.7	3.7	1826	0.2	1894	9.5	62
June	370 1	3.7	1885	0.7	1867	11.0	71
July	419.0	4 2	1894	0.8	1842	11 8	76
Aug.	452 5	4.5	1821	0.4	1867	15.8	74
Sept.	353.1	3.5	1846, 1884	0.2	1882	12.1	67
Oct.	322.8	3.2	1892	0.3	1833	10.0	57
Nov.	329.2	3.3	1890, 1908, 1917	0.8	1846	8.0	45
Dec.	350.7	3.5	1828	0.3	1823	7.4	36
Year	4322.1	42.8	1825	29.7	1867	61.2	54

Lowest 5 consec. months, July–Nov., 1881, 8.1.
Highest 7 consec. months, Feb.–Aug., 1867, 47.7.
Extent of record, 101 yrs., 1820–1920. Lowest Jan. temp., –5° (43 yrs.).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	BALTIMORE, Md. Elev. 123 ft.						
	Rainfall, inches						Mean temp., °F. (43 yrs.) (1878–1920)
Time	Totals, 104 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	302.8	2.9	1825, 1862	0.6	1859	7.1	33
Feb......	316.7	3.0	1864	0.1	1820	8.2	35
Mar......	378.2	3.6	1910	0.5	1829	9.1	42
Apr......	333.7	3.2	1847, 1855	0.4	1839	9.1	53
May.....	356.5	3.4	1826, 1866	0.2	1858	9.1	64
June.....	372.6	3.6	1853	0.6	1836	9.2	73
July......	421.9	4.0	1864, 1870	0.4	1889	11.0	77
Aug......	420.2	4.0	1821, 1844	0.3	1817	10.0	75
Sept.....	353.8	3.4	1884	0.1	1821	10.7	68
Oct......	305.8	2.9	1874, 1920	0.2	1833	7.9	58
Nov......	305.0	2.9	1870	0.3	1852	7.9	46
Dec......	340.1	3.3	1828	0.3	1839	8.8	36
Year...	4208.2	40.4	1870	22.4	1889	62.4	55

Lowest 5 consec. months, Feb.–June, 1856, 6.6. Extent of record, 104 yrs., 1817–1920.
Highest 7 consec. months. Mar.–Sept., 1889, 44.4. Lowest Jan. temp., −6° (49 yrs.).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	WASHINGTON, D. C. Elev. 112 ft.								
	Rainfall, inches								Mean temp., °F. (45 yrs.) (1876–1920)
Time	Totals	Average	No. in averages	Missing years	Minimum		Maximum		
					Year	Amount	Year	Amount	
Jan...	256.5	3.2	81	1830–38, 40, 43–45, 49–51	1872	0.2	1882	7.1	33
Feb...	245.3	3.0	82	1830–38, 43–45, 49–51	1864	0.3	1884	6.8	35
Mar...	283.9	3.5	82	1830–38, 43–45, 49–51	1910	0.6	1867	9.2	42
Apr....	272.2	3.3	82	1830–38, 43–45, 49–51	1847	0.3	1889	9.1	53
May...	291.4	3.6	82	1830–38, 43–45, 49–51	1826	0.8	1889	10.7	64
June...	317.6	3.9	82	1830–38, 43–45, 49–51	1864	0.8	1900	10.9	72
July...	356.1	4.4	81	1827, 30–37, 42–45, 49–51	1872	0.8	1886	10.6	77
Aug...	324.9	4.0	81	1827, 30–37, 42–45, 49–51	1854	0.6	1906	14.4	75
Sept...	254.7	3.1	81	1827, 30–37, 42–45, 49–51	1884	0.1	1876	10.8	68
Oct....	249.6	3.0	82	1830–37, 42–45, 49–51	1874, 1892, 1895	0.3	1866	10.6	57
Nov...	205.9	2.5	82	1830–37, 42–45, 49–51	1828	0.0	1877	7.2	45
Dec...	249.5	3.0	82	1830–37, 42–45, 49–51	1889	0.2	1901	7.6	36
Year	3292.7	40.6	81	1827, 30–35, 38, 40, 42–45, 49–51	1826	18.8	1878	62.1	55

Lowest 5 consec. months, Aug.–Dec., 1828, 4.2. Extent of record, 1824–1920.
, 46.4. Lowest Jan. temp., −14° (48 yrs.).

Selected Long-term Rain and Temperature Records.—*(Continued)*

Place	ATLANTA, Ga. Elev. 1174 ft.						
	Rainfall, inches						Mean temp., °F. (54 yrs.) (1867–1920)
Time	Totals, 57 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	265.4	4.7	1876	1.2	1883	15.8	43
Feb......	275.3	5.0	1898, 1906	0.6	1873	12.0	45
Mar......	309.8	5.5	1905, 1918	0.9	1880	11.9	52
Apr......	221.0	4.0	1915	0.4	1874	10.4	61
May.....	192.2	3.5	1897, 1914	0.3	1871	7.8	70
June.....	215.6	3.9	1868	0.5	1912	11.2	76
July......	241.4	4.4	1881	0.6	1887	14.1	78
Aug......	246.8	4.5	1877	1.0	1874, 1920	10.0	76
Sept.....	183.7	3.3	1884, 1897	0.1	1888	14.3	72
Oct......	139.8	2.5	1886, 1891	0.0	1868	8.9	62
Nov......	178.6	3.2	1890	0.2	1880	8.2	52
Dec......	267.8	4.7	1889	0.6	1919	12.9	43
Year...	2710.9	49.3	1904	33.1	1920	65.3	61

Lowest 5 consec. months, July–Nov., 1884, 8.1.
Highest 7 consec. months, Sept., 1880–Mar., 1881, 52.7.
Extent of record, 57 yrs., 1859, 1865–1920. Missing: Jan.–Oct., 1865; Apr.-Nov. 1867, and corresponding totals.
Lowest Jan. temp., –2° (44 yrs.).

Selected Long-term Rain and Temperature Records.—*(Continued)*

Place	PITTSBURGH, Pa. Elev. 842 ft.						
	Rainfall, inches						Mean temp., °F. (43 yrs.) (1878–1920)
Time	Totals, 81 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	213.5	2.7	1851, 1859	0.4	1888	6.2	31
Feb......	190.4	2.4	1841	0.1	1887	6.5	31
Mar......	231.9	2.9	1910	0.4	1898	5.4	39
Apr......	236.5	3.0	1849	0.8	1852	9.3	51
May.....	256.0	3.3	1911	0.4	1858	6.6	62
June.....	286.1	3.7	1894	0.6	1855	7.6	71
July......	297.6	3.9	1894, 1909	1.2	1887	9.5	74
Aug......	259.0	3.3	1894	0.4	1864	8.3	72
Sept.....	219.5	2.8	1908, 1914	0.7	1864	8.2	66
Oct......	209.1	2.7	1874, 1897	0.1	1873	6.2	55
Nov......	192.4	2.4	1904	0.2	1855, 1897	5.1	43
Dec......	228.9	2.9	1861	0.4	1890	5.6	35
Year...	2688.6	35.8	1839	25.3	1890	50.6	53

Lowest 5 consec. months, Aug.–Dec., 1908, 6.8.
Highest 7 consec. months, Mar.–Sept., 1865, 38.3.
Extent of record, 81 yrs., 1836–1867, 1872–1920. Missing: Jan.–July, 1836; Apr.–Dec., 1838; Jan.–Aug., 1866; May–Dec., 1867, and corresponding totals.
Lowest Jan. temp., –12° (44 yrs.).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	CHICAGO, Ill. Elev. 823 ft.						
	Rainfall, inches						Mean temp., °F. (46 yrs.) (1875–1920)
Time	Totals, 50 yrs.	Average	Minimum Year	Minimum Amount	Maximum Year	Maximum Amount	
Jan.	104.5	2.1	1919	0.2	1916	4.8	24
Feb.	107.1	2.1	1877, 1920	0.1	1881	6.0	25
Mar.	125.5	2.5	1910	0.3	1877	5.4	35
Apr.	141.3	2.8	1899	0.1	1909	7.7	46
May	185.0	3.7	1897	0.8	1883	7.3	57
June	170.9	3.4	1904	0.6	1892	10.6	67
July	169.7	3.4	1874, 1894	0.6	1889	9.6	73
Aug.	150.0	3.0	1893	0.2	1885	11.3	71
Sept.	150.8	3.0	1891	0.3	1894	8.3	65
Oct.	127.2	2.5	1897	0.2	1883	7.4	53
Nov.	117.7	2.3	1903, 1904, 1914	0.3	1877	6.1	40
Dec.	101.9	2.0	1896	0.2	1895	6.8	29
Year	1649.7	33.0	1901	24.5	1883	45.9	49

Lowest 5 consec. months, Dec., 1898–Apr., 1899, 5.5.
Highest 7 consec. months, May–Nov., 1883, 33.7.
Extent of record, 50 yrs., 1871–1920.
Lowest Jan. temp., −20° (48 yrs.).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	DETROIT, Mich. Elev. 730 ft.						
	Rainfall, inches						Mean temp., °F. (43 yrs.) (1878–1920)
Time	Totals, 50 yrs.	Average	Minimum Year	Minimum Amount	Maximum Year	Maximum Amount	
Jan.	106.4	2.1	1902	0.6	1874	5.0	24
Feb.	108.3	2.2	1877	0.0	1881	6.4	25
Mar.	120.7	2.4	1910	0.4	1913	5.6	33
Apr.	122.2	2.4	1899	0.5	1880	6.2	45
May	164.8	3.3	1920	0.4	1892	7.8	58
June	184.1	3.7	1895	0.6	1892	8.3	68
July	167.6	3.2	1877	0.0	1878	8.8	72
Aug.	138.4	2.8	1889, 1894	0.2	1877	7.3	70
Sept.	132.9	2.6	1877	0.4	1902	6.5	63
Oct.	122.0	2.4	1892	0.3	1881	6.5	52
Nov.	119.9	2.4	1904	0.2	1891	5.3	39
Dec.	115.2	2.3	1900	0.4	1878	4.8	29
Year	1601.9	32.0	1889	21.1	1880	47.7	44

Lowest 5 consec. months, Sept., 1874–Jan., 1875, 5.4.
Highest 7 consec. months, Apr.–Oct., 1880, 36.5.
Extent of record, 50 yrs., 1871–1920.
Lowest Jan. temp., −16° (46 yrs.).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	DULUTH, Minn. Elev. 1133 ft.						
	Rainfall, inches						Mean temp., °F. (48 yrs.) (1873–1920)
Time	Totals, 50 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan......	52.4	1.0	1904	0.2	1916	3.5	9
Feb......	48.4	1.0	1877	0.1	1884	2.7	13
Mar......	78.5	1.6	1883, 1910, 1912, 1915	0.4	1917	5.0	24
Apr......	102.0	2.0	1873	0.3	1894	5.8	40
May.....	170.3	3.4	1900	0.6	1879	8.0	48
June.....	207.9	4.2	1910	0.1	1874	10.9	57
July......	188.5	3.8	1875	0.5	1909	10.8	65
Aug......	163.8	3.3	1878	0.5	1889	7.8	64
Sept.....	174.0	3.5	1892	0.3	1881	11.5	56
Oct......	126.7	2.5	1895	0.1	1894	5.0	45
Nov......	76.1	1.5	1916	0.1	1919	3.9	30
Dec......	59.6	1.2	1905	0.1	1879	3.9	17
Year...	1446.5	28.9	1910	18.1	1879	45.3	39

Lowest 5 consec. months, Jan.–May, 1900, 2.9.
Highest 7 consec. months, May–Nov., 1879, 36.4.
Extent of record, 50 yrs., 1871–1920.
Lowest Jan. temp., –41° (44 yrs.).

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	ST. PAUL, Minn. Elev. 837 ft.						
	Rainfall, inches						Mean temp., °F. (48 yrs.) (1873–1920)
Time	Totals, 85 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan....	77.5	0.9		0.0	1881	4.3	12
Feb....	68.2	0.8	†	0.0	1869	2.8	15
Mar....	117.0	1.4	*1853	0.0	1849	4.1	28
Apr.....	201.2	2.4	1848	0.2	1860, 1862	5.8	46
May...	288.1	3.4	1900	0.3	1906	10.4	58
June....	343.1	4.1	1863	0.0	1874	11.7	67
July....	302.5	3.6	1894	0.1	1838	11.1	72
Aug....	289.1	3.4	1894	0.4	1849	9.6	69
Sept....	276.4	3.2	1882	0.3	1869	10.6	60
Oct.....	180.3	2.1	1853, 1857	0.0	1900, 1911	7.6	48
Nov.	117.0	1.4	1848, 1904, 1912, 1917	0.1	1857	5.8	32
Dec......	82.3	1.0	1850, 1913	0.0	1857	3.0	19
Year...	2308.7	27.5	1910	10.2	1849	49.7	44

Lowest 5 consec. months, Nov., 1852–Mar., 1853, 0.4.
Highest 7 consec. months, Apr.–Oct., 1849, 40.7.
Extent of record, 85 yrs., 1834–1920. Missing: Jan.–June, and total, 1836.
Lowest Jan. temp., $-41°$ (47 yrs.).

* 1853, 92, 98, †1846, 53, 54, 64, 77.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	NEW ORLEANS,* La. Elev. 51 ft.						
	Rainfall, inches						Mean temp., °F. (59 yrs.) (1862–1920)
Time	Totals, 75 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan.	338.2	4.6	1840	0.1	1881	11.2	54
Feb.	310.1	4.2	1892	0.0	1875	13.8	57
Mar.	332.4	4.5	1916	0.6	1903	14.6	63
Apr.	356.9	4.8	1915	0.0	1883	14.2	69
May	318.6	4.2	1898	0.0	1873	18.7	75
June	406.9	5.4	1907	1.0	1843	14.6	81
July	484.1	6.5	1859	0.9	1869	15.5	82
Aug.	430.7	5.7	1884	0.9	1888	22.7	82
Sept.	339.7	4.5	1839	0.1	1898	13.9	79
Oct.	263.2	3.5	1874	0.0	1869	13.4	70
Nov.	279.0	3.7	1903	0.2	1851	8.3	62
Dec.	352.2	4.7	1889	0.7	1905	14.4	55
Year	4174.0	56.4	1899	31.1	1875	85.7	69

Lowest 5 consec. months, Oct., 1889–Feb., 1890, 6.1.
Highest 7 consec. months, Feb.–Aug., 1888, 63.2.
Extent of record, 75 yrs.: 1836, 1839–1860, 1868, 1870–1920, except Jan.–Mar., and total, 1868.
Lowest Jan. temp., + 15° (59 yrs.).

* For analysis of records, 1894 to 1918, see *E. N. R.*, May 13, 1920, p. 963.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	DENVER, Col. Elev. 5291 ft.						
	Rainfall, inches						Mean temp., °F. (48 yrs.) (1873–1920)
Time	Totals, 49 yrs.	Average	Minimum		Maximum		
			Year	Amount	Year	Amount	
Jan.	21.5	0.4	1914	0.0	1883	2.4	30
Feb.	26.4	0.5	1916	0.0	1909	1.4	32
Mar.	48.1	1.0	1908	0.1	1891, 1905	3.1	40
Apr.	103.2	2.1	1878	0.1	1900	8.2	48
May	121.2	2.5	1886	0.1	1876	8.6	56
June	65.7	1.3	1890	0.0	1882	5.0	67
July	87.5	1.8	1901	0.0	1919	5.2	72
Aug.	67.0	1.4	1900, 1917	0.1	1908	3.2	71
Sept.	50.9	1.0	1879, 1892	0.0	1909	3.8	63
Oct.	48.9	1.0	1876	0.1	1892	3.9	51
Nov.	27.6	0.6	1899, 1901, 1917	0.0	1886	1.9	40
Dec.	34.4	0.7	*	0.0	1913	5.2	32
Year	701.7	14.3	1911	7.8	1909	23.0	51

Lowest 5 consec. months, Sept., 1879–Jan., 1880, 1.1.
Highest 7 consec. months, Mar.–Sept., 1909, 19.1
Extent of record, 49 yrs., 1872–1920.
Lowest Jan. temp., −29° (49 yrs.).

* 1881, 90, 95, 1905, 6.

Selected Long-term Rain and Temperature Records.—*(Continued)*

Place	FORT DAVIS, Texas. Elev. 4927 ft.						
	Rainfall, inches						Mean temp., °F. (18 yrs.)
			Minimum		Maximum		
Time	Totals, 31–35 yrs.	Average	Year	Amount	Year	Amount	
Jan......	17.8	0.5	A	0.0	1858	3.0	44
Feb......	16.6	0.5	A	0.0	1877	3.5	48
Mar......	14.0	0.4	A	0.0	1905	2.6	54
Apr......	17.4	0.5	A	0.0	1855	3.6	62
May.....	34.5	1.0	A	0.0	1881	6.3	69
June.....	66.3	2.0	A	0.0	1873	6.8	74
July......	108.2	3.2	1903	0.2	1875	15.4	75
Aug......	122.5	3.6	1871	0.2	1876	10.4	73
Sept.....	100.4	3.0	1869, 1871	0.1	1887	7.1	68
Oct......	45.7	1.3	A	0.0	1879	6.2	61
Nov......	18.9	0.6	A	0.0	1905	3.2	50
Dec......	19.5	0.6	A	0.0	1874	4.0	45
Year...	558.1	18.0	1871	6.8	1875, 1876	27.7	60

Lowest 5 consec. months, 0.0. A. Several years.
Highest 7 consec. months, Mar.–Sept., 1875, 25.6.
Extent of record, 1855–1907, 53 yrs., omitting 1861–1868; Jan.–May, and total, 1869; July–Dec., and total, 1891; 1892–1901; Apr., May, June, Nov., Dec., and total, 1906; Jan., Apr., Nov., Dec., and total, 1907.
Lowest Jan., temp. −3° (16 yrs.).

Selected Long-term Rain and Temperature Records.—*(Continued)*

Place	PHOENIX, Ariz.* Elev. 1108 ft.						
	Rainfall, inches						Mean temp., °F. (25 yrs.) (1896–1920)
			Minimum		Maximum		
Time	Totals, 45 yrs.	Average	Year	Amount	Year	Amount	
Jan......	38.4	0.9	A	0.0	1897	3.7	51
Feb......	39.1	0.9	A	0.0	1905	4.6	56
Mar......	29.0	0.7	A	0.0	1905	2.4	59
Apr......	16.3	0.4	A	0.0	1905	2.6	66
May.....	5.5	0.1	A	0.0	1893	1.0	74
June.....	3.4	0.1	A	0.0	1899	0.8	85
July......	49.9	1.2	A	0.0	1911	6.5	89
Aug......	42.3	1.0	A	0.0	1918	3.5	88
Sept.....	29.4	0.7	A	0.0	1897	3.7	82
Oct......	19.3	0.4	A	0.0	1911	2.2	70
Nov......	30.3	0.7	A	0.0	1905	3.6	60
Dec......	38.6	0.9	A	0.0	1883, 1889	3.4	51
Year...	326.2	7.8	1885	3.8	1905	19.7	70

Lowest 5 consec. months, 0.0. A. Several years.
Highest 7 consec. months, Dec., 1904–June, 1905, 13.6.
Extent of record, 45 yrs., 1876–1920. Missing: Jan., June–Aug., and total, 1876; July–Dec., and total, 1887: Jan.–Mar., Oct., Dec., and total, 1888.
Lowest Jan. temp., +12° (27 yrs.).

* See also Table 6, p. 21.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	LOS ANGELES, Cal. Elev. 338 ft.*						
	Rainfall, inches						Mean temp., °F. (17 yrs.) (1904–1920)
			Minimum		Maximum		
Time	Totals, 44 yrs.	Average	Year	Amount	Year	Amount	
Jan......	140.5	3.3	1904, 1912	0.1	1916	13.3	55
Feb......	136.6	3.2	A	0.0	1884	13.4	56
Mar......	127.2	3.0	A	0.0	1884	12.4	58
Apr......	38.9	0.9	A	0.0	1880	5.1	60
May......	18.0	0.4	A	0.0	1892	2.1	61
June......	3.1	0.1	A	0.0	1884	1.4	66
July......	0.6	0.01	A	0.0	1886	0.2	70
Aug......	1.0	0.02	A	0.0	1889	0.3	71
Sept......	7.4	0.2	A	0.0	1919	1.3	59
Oct......	30.4	0.7	A	0.0	1889	7.0	66
Nov......	53.5	1.2	A	0.0	1900	6.5	62
Dec......	115.8	2.6	A	0.0	1889	15.8	57
Year...	736.5	17.1	1898	4.8	1884	40.2	63

Lowest 5 consec. months, 0.0. A. Several years.
Highest 7 consec. months, Dec., 1883–June, 1884, 37.0.
Extent of record, 4.4 yrs., 1877–1920. Missing: Jan.–June, and total, 1877.
Lowest Jan. temp., + 28° (35 yrs.).

* See also p. 22.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	SAN FRANCISCO, Cal. Elev. 155 ft.*							
	Rainfall, inches						Mean temp., °F.	
			Minimum		Maximum			
Time	Totals, 71 yrs.	Average	Year	Amount	Year	Amount	High	Low
Jan......	349.2	4.9	1920	0.3	1862	24.4	73	29
Feb......	260.1	3.7	1864	0.0	1878	12.5	80	33
Mar......	230.5	3.2	1898	0.2	1879	8.8	80	33
Apr......	108.2	1.5	A	0.0	1880	10.1	88	40
May......	50.6	0.7	A	0.0	1883	3.5	97	42
June......	10.2	0.1	A	0.0	1884	2.6	100	46
July......	1.0	0.01	A	0.0	1860, 1886	0.2	93	47
Aug......	1.2	0.02	A	0.0	1916	0.3	92	46
Sept......	23.0	0.3	A	0.0	1904	5.1	101	47
Oct......	64.4	0.9	A	0.0	1889	7.3	94	45
Nov......	173.4	2.4	A	0.0	1885	11.8	93	38
Dec......	318.3	4.5	1876	0.0	1866	15.2	72	34
Year..	1589.5	22.4	1917	8.9	1884	38.8	57	

Lowest 5 consec. months, 0.0. A., Several years.
Highest 7 consec. months, Nov., 1861–May, 1862, 49.1.
Extent of record, 71 yrs., 1850–1920.
Lowest Jan. temp., +29°.

* See also p. 22.

Selected Long-term Rain and Temperature Records.—(*Continued*)

Place	SEATTLE, Wash. Elev. 123 ft.						
	Rainfall, inches						Mean temp., °F. (29 yrs.) (1892–1920)
Time	Totals, 29 yrs.	Average	Minimum Year	Minimum Amount	Maximum Year	Maximum Amount	
Jan......	137.5	4.7	1898, 1917	2.0	1914	9.8	39
Feb......	106.3	3.7	1920	0.3	1902	8.1	39
Mar......	82.7	2.9	1906, 1911	0.9	1894, 1904	6.2	44
Apr......	71.2	2.5	1906	0.3	1893	5.5	49
May.....	56.6	1.9	1904	0.3	1893	4.3	55
June.....	48.3	1.7	1908	0.2	1919	5.4	60
July......	19.6	0.7	1896, 1910, 1914	0.0	1897	2.4	63
Aug......	14.7	0.5	1894	0.0	1899	2.5	63
Sept.....	51.1	1.8	1918	0.1	1907	3.4	58
Oct......	77.6	2.7	1895	0.0	1898	4.7	51
Nov......	169.3	5.8	1895	2.0	1896	9.5	45
Dec......	153.4	5.3	1914	1.4	1897	11.8	41
Year...	983.1	33.9	1911	21.7	1902	45.8	51

Lowest 5 consec. months, June–Oct., 1895, 1.9.
Highest 7 consec. months, Oct., 1893–Apr., 1894, 37.4.
Extent of record, 29 yrs., 1892–1920.
Lowest Jan. temp., +3°.

Table 6. Temperature and Rainfall in Desert Region, Southwest United States
U. S. Weather Bureau

Place...	Yuma, Ariz.			Phoenix, Ariz.			Tucson, Ariz.			Greenland* Ranch, Calif.		
Extent of record, years.........	45	45	31	42	42	32	42	42	52	9	9	9
	Temperature		Mean rainfall, in.	Temperature		Mean rainfall, in.	Temperature		Mean rainfall, in.	Temperature		Mean rainfall, in.
Month	Max.	Min.		Max.	Min.		Max	Min.		Max.	Min.	
Jan.............	84	22	0.4	87	12	1.2	90	6	0.8	85	15	0.4
Feb.............	92	25	0.6	92	19	0.7	91	11	0.9	90	28	0.4
March..........	100	31	0.4	97	24	0.5	95	22	0.7	98	30	0.3
April...........	107	38	0.1	105	30	0.4	100	28	0.3	109	36	0.0
May............	120	39	0.0	114	35	0.0	111	32	0.2	120	48	0.1
June............	119	50	0.0	119	33	0.1	112	37	0.2	124	55	0.1
July.............	118	61	0.1	117	46	1.1	111	49	2.6	134	67	0.1
Aug.............	117	58	0.4	116	49	1.0	110	55	2.5	126	68	0.0
Sept	113	50	0.2	114	39	1.0	107	43	1.1	116	54	0.0
Oct.............	108	38	0.2	105	34	0.4	101	27	0.6	106	40	0.1
Nov............	94	29	0.3	97	24	1.0	93	21	0.8	91	30	0.2
Dec.............	83	22	0.4	95	18	0.6	90	10	1.2	82	21	0.1
Year....... ..	120	22	3.1	119	12	7.9	112	6	11.8	134	15	1.7

* Greenland Ranch at mouth of Furnace Creek, Death Valley, Cal. 178 ft. below sea-level.
The highest temperature on record in the United States, to include 1920, was 134 at Greenland Ranch, Cal.

Records Outside of United States. At Rancagua, Chili, annual rainfall 1910–1916, varied from 15.8 to 60.6 in.[11] Rainfall in Hawaii varies annually from a few inches to nearly 600; it varies greatly in different localities.[12] Rainfall at Victoria reservoir, Western Australia, from 1897 to 1915 varied

Table 7. Temperatures and Rainfall in California, 1883–1920

U. S. Weather Bureau

Year	Red Bluff				Sacramento				Fresno				Los Angeles*				San Francisco*			
	Temperature			Rainfall	Temperature			Rainfall	Temperature			Rainfall	Temperature			Rainfall	Temperature			Rainfall
	Mean	Max.	Min.		Mean	Max.	Min.		Mean	Max.	Min.		Mean	Max.	Min.		Mean	Max.	Min.	
1883	61.5	107	19	13.8	58.8	104	22	13.5	.....	.....	.....	.....	61.6	104	28	14.1	54.7	95	35	15.4
1884	60.8	107	22	28.1	59.8	100	21	34.9	.....	.....	.....	.....	60.8	102	34	40.2	56.7	83	35	38.8
1885	64.4	108	33	29.6	62.5	105	34	20.7	.....	.....	.....	.....	63.0	108	36	10.5	57.8	87	43	24.9
1886	63.2	109	30	17.2	60.3	105	27	18.2	.....	.....	.....	.....	61.1	98	32	16.7	57.3	94	41	20.0
1887	64.4	112	27	13.6	60.3	100	28	13.4	.....	.....	.....	.....	61.7	100	33	16.0	55.3	97	33	19.0
1888	64.0	109	18	24.9	61.4	108	19	18.5	64.6	111	20	8.8	60.7	99	31	20.8	56.7	93	29	23.0
1889	63.3	111	26	32.9	60.9	104	31	27.5	64.3	112	27	12.3	63.0	103	32	33.3	57.9	89	39	36.9
1890	61.5	110	22	25.5	59.4	102	29	21.0	62.6	111	24	8.4	63.6	105	34	12.7	56.3	86	36	25.4
1891	62.4	114	26	23.0	60.6	106	26	15.6	63.0	114	26	8.9	63.0	109	33	12.8	56.6	100	37	21.1
1892	62.2	108	28	33.5	60.2	106	26	23.6	63.1	112	27	8.8	61.6	99	35	18.7	56.0	92	38	22.1
1893	60.6	106	27	24.4	58.8	103	28	16.6	61.0	109	28	9.4	61.5	92	31	22.0	54.3	90	36	17.9
1894	62.0	110	27	27.0	60.3	108	26	22.6	62.6	109	25	12.5	60.3	99	32	7.5	55.1	94	36	24.3
1895	62.2	108	28	22.6	60.2	102	28	17.4	62.5	110	26	10.4	61.7	100	34	12.6	55.6	89	38	17.1
1896	62.5	109	26	28.2	60.7	104	28	25.1	63.7	111	28	11.0	63.1	103	35	11.8	55.9	91	33	28.3
1897	62.0	109	27	20.1	59.8	105	28	15.3	62.3	110	23	8.4	61.8	97	30	14.3	55.0	92	38	16.4
1898	62.5	112	24	12.9	59.5	110	26	10.0	63.1	114	24	5.0	62.5	99	31	4.8	54.6	89	36	9.3
1899	62.4	109	26	28.8	59.6	102	30	21.1	62.6	111	24	10.5	62.0	100	33	8.7	55.0	94	34	23.2
1900	62.4	109	29	21.8	59.9	102	30	17.9	62.8	109	26	11.1	63.5	96	37	11.3	56.2	92	40	15.3
1901	63.1	111	25	25.5	60.1	105	26	18.5	63.5	110	27	8.1	62.1	97	31	12.0	55.2	91	37	19.8
1902	61.4	115	28	35.5	59.2	107	29	17.9	62.2	110	24	7.9	61.2	94	32	13.1	55.4	83	38	19.2
1903	61.5	108	27	22.9	59.4	102	29	14.7	62.2	108	25	6.2	62.2	97	32	14.8	55.3	96	37	18.3
1904	62.8	108	29	33.9	60.1	102	32	21.0	63.7	109	28	13.2	63.9	97	35	11.9	56.4	101	38	24.7
1905	62.9	113	25	23.4	59.7	110	28	15.0	63.0	115	26	7.3	62.6	101	33	19.2	56.3	98	39	16.2
1906	63.0	110	30	40.4	60.4	104	30	30.7	63.2	111	27	16.1	62.9	105	34	21.5	56.2	86	40	26.3
1907	61.2	101	27	24.7	59.6	99	31	20.1	62.5	103	30	9.0	62.6	103	35	15.3	56.0	90	36	22.5
1908	62.3	114	25	17.9	59.7	103	28	11.2	63.0	114	29	7.1	61.9	96	37	13.7	55.0	89	35	16.4
1909	62.1	106	29	35.0	59.7	101	29	24.9	62.8	107	30	16.5	61.4	103	37	23.9	55.2	94	38	31.4
1910	62.9	111	26	14.6	59.9	103	28	7.8	64.4	110	25	4.9	63.1	100	33	4.9	55.2	90	36	12.4
1911	61.1	112	26	22.7	58.4	100	30	21.1	62.3	111	24	11.2	62.2	95	38	17.8	55.2	87	38	26.0
1912	60.8	109	26	21.8	59.3	103	29	11.0	62.3	109	24	7.3	62.6	100	38	9.8	56.6	94	40	15.6
1913	62.5	110	25	25.8	60.4	109	26	14.3	63.4	109	17	8.7	62.5	108	28	17.2	56.4	101	33	19.0
1914	62.4	108	28	27.5	59.8	102	30	16.0	63.6	106	27	9.7	63.2	97	40	23.2	56.7	92	41	24.0
1915	62.2	110	28	36.2	60.4	105	24	17.7	63.4	109	24	11.6	63.0	99	38	16.7	57.4	86	36	28.3
1916	61.3	113	27	20.1	59.5	105	30	18.3	62.1	108	28	12.5	60.9	96	36	23.3	55.9	87	37	28.1
1917	63.2	110	24	14.2	60.7	107	26	8.9	63.5	109	25	3.9	63.0	105	38	8.4	56.3	96	34	9.0
1918	62.4	112	26	23.6	59.9	107	29	16.9	63.2	107	25	13.7	63.9	100	37	17.5	56.8	85	38	20.8
1919	61.9	108	24	15.7	59.3	107	24	12.8	63.1	108	26	4.5	62.9	96	36	8.8	55.8	93	37	19.0
1920	61.5	110	28	22.0	59.2	108	29	14.8	62.5	110	26	9.8	62.4	92	41	11.2	56.1	92	38	18.3

Note. Red Bluff represents northern extremity of Sacramento Valley; Sacramento, central of that valley; San Francisco, central coast region; Fresno, central of San Joaquin Valley; Los Angeles, Southern California coast. Temperature, degrees Fahrenheit; rainfall, inches.

* See also page 20.

yearly from 18.3 to 44.2, averaging 35.9.[13] At Havana, from 1859 to 1914,[14] annual rate varied from 28.7 to 64.5, averaging 41.7. Rainfall at Cherrapunji, India, varied from 260.21 to 586.6 in.[15] For Chinese averages, see Freeman, *T. A. S. C. E.*, Vol. 85, 1922, p. 1456.

Table 8. Period within Which 50 Per Cent. of Annual Precipitation Occurs

Regions	Period
New York Lake Region	June 20–Dec. 10
Central Virginia	May 1–Oct. 1
Northern Florida	May 15–Oct. 15
Ohio	Mar. 1–Aug. 5
Missouri	May 5–Sept. 25
Gulf Coast	Dec. 1–May 5
Northern Great Plains	May 1–Aug. 10
Southern Great Plains	Apr. 15–Sept. 20
Snake River Region	Nov. 15–Mar. 20
Arizona	July 1–Oct. 20
Willamette Valley	Nov. 1–Feb. 5
Central California	Dec. 20–Mar. 5

From data compiled by U. S. Weather Bureau from 1600 Stations (1895–1914) and 2000 shorter records in "Atlas of American Agriculture" (1923).

Table 9. Range of Rainfall, Foreign Stations[19]

Station	Length of record in years	Annual rainfall in inches	
		Maximum	Minimum
San Diego, Chile	65	32.3	3.4
Mondidier, France	86	32.7	13.9
Berlin, Germany	50	30.2	14.2
Warsaw, Poland	69	46.6	14.6
Calcutta, India	71	98.48	38.43
Bombay, India	54	114.89	35.90
Barnaul, Siberia	45	17.6	4.2
Peking, China	31	41.9	9.5
Hakodate, Japan	33	57.1	27.7
Adelaide, S. Australia	69	30.87	13.43
Brisbane, Queensland	59	88.26	16.17
Cape Town, S. Africa	73	41.03	17.71

Droughts. In Central States, previous to 1876, Iowa State Weather Service reports the greatest drought as 133 days in 1763. The year 1913 was hot and dry in all parts of United States east of Rockies; at Clay Center, Kan., in 70 days of the growing season, rainfall was but 0.03 in. An observer at Ottawa, Kan., reported that the river was the lowest in 53 years.

In Boston, a 36-day drought was ended on Apr. 3, 1915; on Nov. 22, 1924, a 44-day drought ended. In August and September, 1874, a 25-day drought occurred. The driest recent years have been 1894, 1901, 1911, 1913, and 1923. From June 1 to Sept. 30, 1923, the rainfall in Eastern United States was 3.83 in. (25 per cent.) below normal. In 1921,[18] there was a drought in British Isles, surpassing those of 1864 and 1898; from Jan. 1 to Nov. 30, rainfall in London was 46.7 per cent. of normal; water supplies were endangered. A graphical study of droughts is given by V. V. Tchikoff in *E. N. R.*, Sept. 18, 1919, p. 554. A table in *Monthly Weather*

Review, November, 1914, p. 630, indicates that driest periods at New York (1871–1914) fall in September and October. For 28 days in September 1874 only three traces are recorded. J. B. Kincer presents a drought map for summer months in *Monthly Weather Review*, September, 1919, p. 630; historic droughts in United States are described in *Monthly Weather Review*, Vol. 26, 1898, p. 262. The following articles are from *Monthly Weather Review:* Kansas drought, 1910 and 1911, Vol. 38, 1910, p. 1704, and Vol. 39, 1911, p. 1383; South Carolina, 1910–1911, Vol. 39, 1911, p. 664; New York City, 1871–1914, Vol. 42, 1914, p. 629; California, Vol. 48, 1920, p. 156.

MAXIMA RATES* OF RAINFALL

Studies of Miami Conservancy District, Ohio. Data on great storms and frequency of high rates of rainfall for eastern U. S. are given in Part V, *Technical Reports*.† Diagrams indicate that once in 100 years, a rainfall rate per 24 hours may be expected varying from 5 in. along the Great Lakes and St. Lawrence River to 10 in. along the Gulf of Mexico, except for a small area in Texas, which might be subject to 11 in.; for a 50-year period the limits become

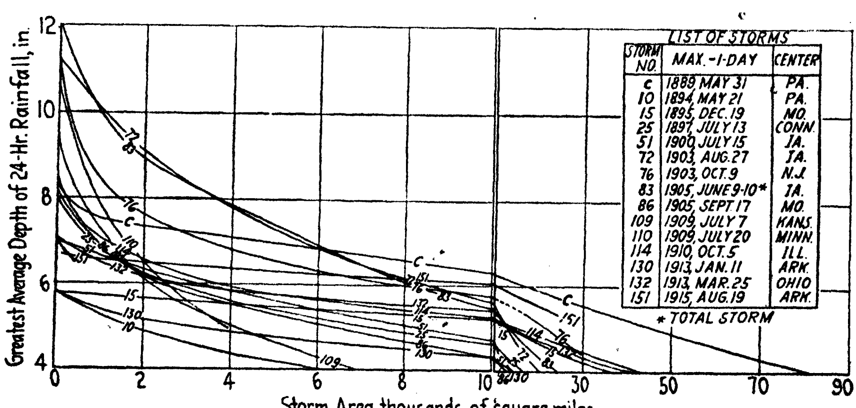


FIG. 3.—Time-area-depth curves for storms over northern states showing greatest average depth of rainfall during 1 day.

respectively 4 and 9 in. Figure 3 was derived from 15 great storms between 1889 and 1915, and indicates the risk in proportioning a spillway for a 6-in. run-off from the watershed (see p. 93).

Curves on Fig. 4 are based on curves, *T. A. S. C. E.*, Vol. 54, 1905. On Fig. 4 have been introduced Allen's New York curves,[16] Sherman's Springfield curves,[17] and Grunsky's San Francisco curves.[5b] U. S. Housing Corporation,

* See also "Excessive Storms, U. S., 1910–1918," C. W. Sherman, *E. N. R.*, May 29, 1919, p. 1065; and tabulation of European, Indian, and American storms in *Proc. Inst. C. E.*, Vol. 217, 1924, p. 284.

† Hazen discusses this data in *E. N. R.*, Nov. 24, 1921, p. 858.

John W. Alvord, Chief Engineer, prepared curves for 23 localities for storms during 1896–1917.*

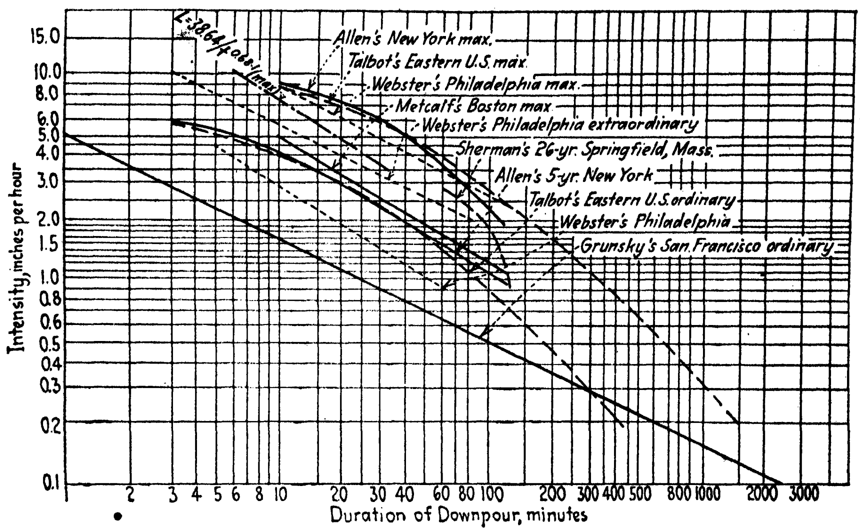


FIG. 4.—Relation between intensity and duration of rainfall.

Bibliography, Chapter 1, Rainfall

1. 1st *Ann. Rept.*, 1911, p. 155. **2.** "Elements of Hydrology," by A. F. Meyer (Wiley, 1917). **a**, p. 81; **b**, p. 417; **c**, p. 87. **3.** "Conservation of Water by Storage," (Yale Press, 1915). **4.** *E. N. R.*, Mar. 28, 1918, p. 628. **5.** *T. A. S. C. E.*, Vol. 85, 1922. **a**, p. 68; **b**, p. 72. **6.** *J. N. E. W. W. A.*, Vol. 33, 1919, p. 74. **7.** *Min. Proc. Inst. C. E.*, Vol. 39, 1874, p. 17; and Vol. 109, 1892, p. 128; also "Rainfall, Reservoirs, and Water Supply," (Van Nostrand, 1913). **8.** *J. A. W. W. A.*, Vol. 3, 1916, p. 1. **9.** *E. N.*, Jan. 6, 1916, p. 4. **10.** *E. N.*, Mar. 29, 1917, p. 503. **11.** *E. N. R.*, May 24, 1917, p. 410. **12.** *T. A. S. C. E.*, Vol. 87, 1924, p. 461, and *E. N. R.*, June 10, 1920, p. 1166. **13.** *4th Report*, Western Australia Water Supply Dept., 1916. **14.** *E. N.*, Oct. 28, 1915, p. 830. **15.** Horton, *J. N. E. W. W. A.*, Vol. 33, 1919, p. 65. **16.** *E. N. R.*, Apr. 7, 1921, p. 589. **17.** *E. N. R.*, Sept. 2, 1920, p. 445. **18.** *E. N. R.*, Nov. 17, 1921, p. 819; and Feb. 2, 1922, p. 204. **19.** *Water Power*, February, 1921, p. 17. **20.** *E. N. R.*, Aug. 2, 1917, p. 212. **21.** *T. A. S. C. E.*, Vol. 84, 1921, p. 225. **22.** "Descriptive Meteorology," (Appleton, 1920), p. 209.

* Four are reproduced in *Munic. J.*, Sept. 14, 1918, p. 205; for New York curve see *T. A. S. C. E.*, Vol. 85, 1922, p. 121.

CHAPTER 2

EVAPORATION*

Value of Data. Evaporation data are of more use to the irrigation engineer, although the waterworks engineer utilizes them in correcting available capacities of storage reservoirs and in computing run-off from rainfall data. The engineer is interested in monthly and yearly data; many meteorological discussions consider days.

Terms. Evaporation is used to denote both the process of vaporization and the quantity of water vaporized and diffused into the atmosphere from land and water surfaces, and frequently includes transpiration, deep seepage, and other losses.[1a]

Effect of Humidity. The amount of evaporation from the free water surfaces of arid regions is considerably greater, especially in warm countries, than evaporation from the sea, situated in the same latitude, since the air possesses much lower relative humidity over the heated lands than over the sea. In the temperate zones, the values of relative humidity inland and at sea approach each other.

Evaporation depends on: (*a*) Quantity of rainfall. (*b*) Distribution of rainfall. (*c*) Extent of the water surface. (*d*) Temperature. (*e*) Barometric effect, which is slight outside of effect of altitude on pressure. A liquid may exist at any temperature only when the pressure upon it is greater than the pressure of its vapor at that temperature. Tate says: "Others things being equal, the evaporation is nearly inversely proportional to the atmospheric pressure." (*f*) Mean daily atmospheric pressure. (*g*) Mean annual atmospheric pressure. (*h*) Wind movement. (*i*) Inclination and geological character of the watershed. (*j*) Extent of forest area, including sprout land. (*k*) Extent of swampy and marshy land. (*l*) Extent of cultivated land. (*m*) Humidity. (*n*) Extent of watershed. Temperature is one of the most important.

Laws of Evaporation. There are 14 general laws of evaporation. 1. For rainfall distributed uniformly throughout the year, evaporation increases proportionally with rainfall. Distribution in showers, downpours, long drizzling rains, and depths and promptness of melting of snowfalls, are important. 2. Heavy winter snow and light summer rainfall together produce a small annual evaporation, and conversely. 3. The greater the watershed, the greater will be the evaporation. 4. The greater the area of water surface on the watershed, the greater will be the evaporation. 5. Evaporation varies nearly inversely as the atmospheric pressure, or nearly directly as the altitude of the watershed. (This is questioned by some authorities.) 6. The rate of evaporation is nearly proportional to the difference of temperatures indicated

* For detailed discussion see "Hydrology," by D. W. Meade, pp. 112–154, (McGraw-Hill Book Company, Inc., 1919); and "Elements of Hydrology," by A. F. Meyer, pp. 188–241, (John Wiley & Sons, Inc., 1917).

by the wet bulb and the dry bulb thermometers. 7. The capacity of the atmospheric air for moisture is approximately doubled for each 20°F. increase in atmospheric temperature. From data on Croton, Pequannock, and Sudbury sheds, T. Merriman deduces the law: For each degree increase in temperature, the rainfall evaporated will be increased by very nearly 2 per cent. 8. Evaporation varies nearly directly as the wind movement. 9. Evaporation from a watershed varies approximately inversely as the square root of the sine of the angle of its average inclination. "Average inclination" is the difference in altitude between the highest and the lowest point, divided by the diagonal of a square of area equivalent to the watershed. (Questioned by some authorities.) 10. Evaporation from a watershed varies nearly as the extent of the surface exposed. The extent of the surface exposed is nearly proportional to area of watershed divided by the cosine of the angle of its average inclination. (Also questioned by some authorities.) 11. Time is an important factor in evaporation, and the shorter the time that it takes water to run off, the less will be the evaporation. Evaporation is comparatively slight in the water-courses. Evaporation varies directly as the time, and time in turn varies nearly inversely as the square root of the sine of the angle of inclination. 12. Evaporation varies nearly inversely as the porosity of the materials covering the watershed, or rather as the depth of the surface of saturation. 13. Evaporation varies approximately with the extent of cultivated land on the watershed. 14. Evaporation varies inversely with the extent of forest and sprout area on the watershed.

Sunshine data, tabulated in *U. S. Weather Review:* see specifically *ibid.*, November, 1919.

FORMULAS

Vermeule's Formula. In formulas below, for monthly evaporation, e = monthly evaporation, in.; $f = (0.05t - 1.48)$, where t is the mean monthly temperature, °F., r = monthly rainfall, in. For yearly evaporation, E =

Month	Formula	Month	Formula
Jan.	$e = f(0.27 + 0.10r)$	July	$e = f(3.00 + 0.30r)$
Feb.	$e = f(0.30 + 0.10r)$	Aug.	$e = f(2.62 + 0.25r)$
Mar.	$e = f(0.48 + 0.10r)$	Sept.	$e = f(1.63 + 0.20r)$
Apr.	$e = f(0.87 + 0.10r)$	Oct.	$e = f(0.88 + 0.12r)$
May	$e = f(1.87 + 0.20r)$	Nov.	$e = f(0.66 + 0.10r)$
June	$e = f(2.50 + 0.25r)$	Dec.	$e = f(0.42 + 0.10r)$

$F(15.50 + 0.16R)$, where E = yearly evaporation, in.; $F = (0.05T - 1.48)$; T = mean yearly temperature, °F.; R = yearly rainfall, in. This formula was deduced from New Jersey conditions (1894 *Report*, N. J. Geological Survey), and has the advantage of allowing for the monthly variations in rainfall. Constants are based on mean temperatures of watershed.

FitzGerald's Formula.[2] FitzGerald's complete formula for evaporation is:

$$E_h = [0.014(S - F_a) + 0.0012(S - F_a)^2](1 + 0.67V^{\frac{1}{2}})$$

The simpler formula is sufficiently accurate for most practical purposes:

$$\frac{(S - F_a)(1 + V/2)}{60} \qquad F_a = S - \frac{0.480(T_a - T_e)b}{689 - T_e}$$

b = height of barometer, in. of mercury; E_h = evaporation per hr., in.; S = maximum force of the vapor in in. of mercury, corresponding to temperature of the water; F_a = force of atmospheric vapor, in in. of mercury; V = velocity of the wind, mi. per hr.; T_a = temperature of the air, °C., by dry thermometer; T_e = temperature of evaporation in °C., by wet thermometer. No difference in evaporation is found between a pan in the sun and one in the shade. The depth of water has no other influence on evaporation than that due to its effect on the temperature of the water. Wind factor = $1 + 0.67V^{\frac{2}{3}}$. Ordinary barometric changes are so slight that they may be disregarded so far as influence on natural evaporation goes. For the correct wind factor, measure the velocity near the water surface, not at some distance from it, nor far above it.

Evaporation is influenced by the wind, as shown by the following observations:

Wind velocity, mi. per hr.	0	5	10	15	20	25	30
Relative evaporation observed	1.0	2.2	3.8	4.9	5.7	6.1	6.3
Relative evaporation computed by FitzGerald's formula	1.0	2.8	3.7	4.4	5.0	5.6	6.2

Wind Factor. See paragraph above for FitzGerald's. Russel found $\left(1 + \frac{V}{4}\right)$, for velocities up to 15 or 20 mi. per hr.; Bigelow, $\left(1 + \frac{V}{11}\right)$; A. F. Meyer uses $\left(1 + \frac{V}{10}\right)$. Wind velocities, as recorded by the Weather Bureau (see Table 307, p. 827) are those about 30 ft. above the ground and are about three times more than those at the surface.

Table 10. Evaporation and Temperatures of Water Surfaces. (FitzGerald*)

Month	Average temperature of water, deg., Fahr.	Evaporation, inches	Month	Average temperature of water, deg., Fahr.	Evaporation, inches
January	32.8	1.0	July	72.3	6.0
February	32.4	1.0	August	71.3	5.5
March	36.4	1.7	September	65.6	4.1
April	46.5	3.0	October	53.6	3.2
May	58.6	4.5	November	42.8	2.2
June	67.9	5.5	December	34.3	1.5
			Total for average year	†	39.2

* Based on experiments at Chestnut Hill reservoir, Boston, 1875–1890.[3] The table above and other experiments indicate that each 9°F. change in water temperature corresponds to about 1 in. per month of change in amount of evaporation. Rate of evaporation was measured at 4 places in Maine by U. S. Geological Survey, July 1, 1905 to Nov. 7, 1908, *Water Supply Paper No.* 279. The annual evaporation was substantially 26 in. † Average annual temperature = 51.2°F.

Dalton's Formula.[4a]—Evaporation from the free surface of water is given by the following equation by Dalton:*

$$E_h = 1.80 \times c \times S\left(\frac{1-d}{b}\right)$$

where c = empirical constant which depends on the air circulation over the water.
S = maximum tension of water vapor at the temperature of the evaporating water, in. of mercury.
d = relative humidity of atmosphere to water vapor.
b = reading of barometer, in. of mercury.

* R. E. Horton proposed a modified formula in *E. N. R.*, Apr. 26, 1917, p. 196.

For quiet air, $c = 0.55$; for moderately agitated air, $c = 0.71$; for heavy wind, $c = 0.86$.

Maxima tensions, S, of water vapor for different temperatures are given in Table 12, below. For moderately agitated air at temperature of 50°F., and with barometer at 29.53 in., and with 60 per cent. relative humidity in the air, $E_h = 1.80 \times 0.71 \times 0.36 \left(\frac{1 - 0.6}{29.53}\right) = 0.0062$ in. per hr.

Lueger assumes for the dry period an average evaporation of 0.16 to 0.39 in. per day, according to climate. Other observers have found that in the temperate zone, with a yearly average temperature of 50°F., the yearly evaporation from a free surface is .3 ft. At a temperature of 77°F., the daily evaporation might be 0.4 in. which gives for each sq. mi. of free water surface a daily loss of 929,000 cu. ft.

Temperature and Wind Relation.[4b] In the tropics, the average annual temperature is 25°C. = 77°F.; from 23½ to 40 deg. of north or south latitude, average temperature is 20°C. = 68°F.; from 40 to 50 deg. latitude, 12.5°C. = 54.5°F.; from 50 to 60 deg. latitude, 5°C. = 41°F.; from 60 to 66½ deg. of latitude (polar circles), 2.5°C. = 36.5°F. and from the Polar Circles to the Poles, −2°C. = 28.4°F. If there is normally a strong wind blowing ($c = 0.86$) so that 75 per cent. represents the relative humidity of air adjacent to water,

$$E_h = 1.80 \times 0.86 \times S(1 - 0.75) \div 29.92.$$

Table 11. Relation of Average Yearly Temperature to Yearly Evaporation[4b]

Computed from Dalton's Formula (Barometer at 29.92 in.)

Temperature	Centigrade	25	20	12	5	2.5	−2
	Fahrenheit	77	68	54	41	36	28
Yearly evaporation	mm.	2660	1965	1180	738	630	452
	in.	105	77	46	29	25	18

Within the range of annual variation of temperature prevailing throughout the Northwest United States, the rate of evaporation will vary about 700 to 1200 per cent., due to temperature changes alone (A. F. Meyer[1b]).

Table 12. Maximum Tension of Water Vapor for Different Temperatures[4a]

Temperature °C	Temperature °F	Vapor tension mm., mercury	Vapor tension in., mercury	Temperature °C	Temperature °F	Vapor tension mm., mercury	Vapor tension in., mercury
−20	−4.0	0.927	0.04	16	60.8	13.536	0.53
−18	−0.4	1.100	0.04	18	64.4	15.357	0.60
−15	+5.0	1.400	0.06	20	68.0	17.391	0.68
−12	10.4	1.780	0.07	22	71.6	19.659	0.77
−10	14.0	2.093	0.08	24	75.2	22.184	0.87
− 8	17.6	2.455	0.10	25	77.0	23.550	0.93
− 5	23.0	3.113	0.12	26	78.8	24.988	0.98
− 3	26.6	3.644	0.14	28	82.4	28.101	1.11
0	32.0	4.600	0.18	30	86.0	31.548	1.24
+ 2	35.6	5.302	0.21	32	89.6	35.359	1.39
4	39.2	6.097	0.24	33	91.4	37.411	1.47
5	41.0	6.534	0.26	34	93.2	39.565	1.56
6	42.8	6.988	0.28	35	95.0	41.827	1.65
8	46.4	8.017	0.32	36	96.8	44.201	1.74
10	50.0	9.165	0.36	37	98.6	46.691	1.84
12	53.6	10.457	0.41	38	100.4	49.302	1.94
14	57.2	11.908	0.47	39	102.2	52.039	2.05
15	59.0	12.699	0.50	40	104.0	54.906	2.16

U. S. Weather Bureau Formula.[5] Study of observations at Abbassia, Boston, Fort Collins and Nakuss showed the constants of Dalton's evaporation formula to be very inconsistent; it was inferred, therefore, that the formula is not satisfactory. The following formula, based on observations, is proposed:

$$E_h = Cf(h)E\frac{de}{ds}(1 + Aw)$$

$Cf(h)$ = a variable, a function of the height of pan.
E = vapor pressure, corresponding to the dew point of air.
$\frac{de}{ds}$ = ratio of increase of vapor pressure to increase of temperature, Centigrade.
w = wind velocity, kilometers per hr.
A = a wind constant.

The size of pan makes an insignificant difference in evaporation, under the same local conditions. Duryea and Haehl[6a] have recommended as values of $Cf(h)$ from Salton sea and other tests: 0.042 for 2-ft. pans, 0.036 for 4-ft. pans; 0.031 for 6-ft. pans, and 0.024 for large lake surfaces. Grunsky points out that no formula of Dalton type, or of the type suggested by Professor Bigelow (U. S. Weather Bureau), will meet the requirements of the engineer when called on to determine from weather conditions alone the quantity of evaporation from a sheet of water not yet in existence.[6b]

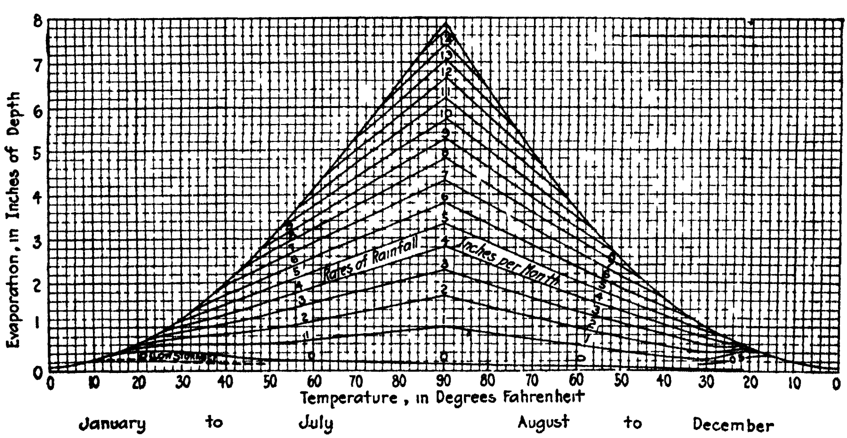


FIG. 5.—Evaporation from land for various temperatures and rainfall rates.[1d]

Meyer's Formula. For evaporation, at St. Paul, Minn.[1c]

$$E_m = 15\,S(1 - d)\left(1 + \frac{V}{10}\right)$$

in which E_m = evaporation, in in. per month,
S = maximum vapor pressure, in in. of mercury, at monthly mean air temperature;
d = relative humidity of atmosphere to water vapor;
V = wind velocity, in mi. per hr., as measured by the Weather Bureau, approximately 30 ft. above the general level of the surrounding country.

Table 13. Evaporation from Water Surfaces in Inches*

Locality (also position of test pan if known)	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	12 Mos.	Year	Remarks, size of test pan and altitude at which placed
Salton sea, Cal., 1500 ft. inland	5.1	7.4	12.5	15.8	19.0	21.5	22.2	18.5	15.5	13.2	7.5	6.4	164.5	1909–10	D = 2, ground
Salton sea, Cal., 1500 ft. inland	7.1	9.4	13.2	17.2	20.5	23.5	25.0	21.2	19.5	17.2	10.0	9.6	193.4	1909–10	D = 2, 40 ft. high
Salton sea, Cal., 500 ft. at sea	3.6	5.0	6.8	9.0	11.0	13.5	14.8	12.5	12.4	9.2	6.2	4.7	108.6	1909–10	D = 4, 2 ft. high
Salton sea, Cal., 500 ft. at sea	5.1	7.2	9.0	10.5	13.0	16.8	18.6	15.0	15.2	12.2	8.1	7.0	137.7	1909–10	D = 4, 45 ft. high
Salton sea, Cal., 7500 ft. at sea	3.4	5.1	7.0	8.8	10.5	13.0	14.0	12.2	12.1	9.2	6.0	5.2	106.4	1909–10	D = 4, 2 ft. high
Salton sea, Cal., 7500 ft. at sea	4.7	7.4	9.5	11.5	14.0	16.8	18.0	15.3	15.4	13.0	7.5	7.0	140.0	1909–10	D = 4, 45 ft. high
Indio, Cal., 15 mi. north of Salton sea	3.2	5.1	7.5	12.0	15.8	16.1	16.3	13.8	12.4	8.9	5.2	3.0	119.3	1909–10	D = 6, ground
Indio, Cal., 15 mi. north of Salton sea	5.5	8.8	12.1	19.2	25.1	26.7	27.2	23.0	21.1	16.8	9.4	5.2	200.4	1909–10	D = 6, 10 ft. high
Mecca, Cal., 0.5 mi. north of Salton sea	2.9	5.0	8.1	10.9	12.7	14.2	15.2	13.2	10.3	8.2	4.1	3.0	107.8	1909–10	D = 6, ground
Mecca, Cal., 0.5 mi. north of Salton sea	5.5	8.8	11.9	17.0	21.3	21.6	22.6	20.4	16.9	12.4	7.2	4.6	170.0	1909–10	D = 6, 10 ft. high
Brawley, Cal., 20 mi. south of Salton sea	3.0	5.0	8.0	10.7	13.8	13.7	14.1	11.3	10.2	7.0	4.1	2.7	103.6	1909–10	D = 6, ground
Brawley, Cal., 20 mi. south of Salton sea	5.3	8.0	11.0	16.0	21.6	20.2	21.0	21.2	16.3	11.6	7.0	4.6	163.7	1909–10	D = 2, 10 ft. high
Mammoth, Cal., 40 mi. sou heast of Salton sea, in desert	4.2	5.7	9.0	12.0	15.5	16.8	18.0	13.7	12.2	9.5	5.3	3.7	125.5	1909–10	D = 6, ground
Mammoth, Cal., 40 mi. sou heast of Salton sea, in desert	6.5	8.9	11.6	17.1	22.0	24.2	25.7	18.2	17.0	14.7	8.1	5.1	179.1	1909–10	D = 2, 10 ft. high
Calexico, Cal.	....	....	....	....	....	....	14.5	12.7	10.3	7.5	4.8	4.5	......	1903	All inland tests. Tanks, 22 to 36 in.; diam., 30 in. deep, set 29 in. in ground. S. Fortier, U. S. Dept. of Agriculture, Bull. 177, 1907.
Calexico, Cal.	4.4	6.3	8.9	9.6	10.9	13.9	12.5	11.0	8.6	8.8	5.4	3.5	103.6	1904	
Calexico, Cal.	2.7	1.7	4.4	4.7	8.4	12.9	10.4	8.5	7.8	6.8	3.2	3.4	75.1	1905	
Pomona, Cal.	....	....	....	....	....	....	9.1	9.4	6.3	6.6	4.2	2.5	......	1903	
Pomona, Cal.	2.8	2.6	3.7	5.0	6.5	8.2	9.3	9.4	7.2	5.4	4.0	2.9	67.0	1904	
Pomona, Cal.	1.9	1.6	3.7	4.1	6.0	7.7	8.9	9.0	7.4	5.3	4.0	2.9	62.8	1905	
Tulare, Cal.	....	....	....	....	....	....	....	12.3	9.1	6.3	3.5	2.0	......	....	
Tulare, Cal.	1.5	3.0	3.8	4.6	8.0	10.7	12.2	12.0	8.7	4.3	3.9	2.0	74.5	1904	
Tulare, Cal.	1.5	2.5	2.9	3.3	6.8	10.3	11.1	7.6	6.3	4.8	3.9	2.0	62.9	1905	
Chico, Cal.	0.5	1.1	1.1	3.0	5.6	6.8	8.3	7.7	4.8	2.2	1.6	1.0	43.5	1904	
Chico, Cal.	0.1	1.2	3.8	4.8	6.8	9.1	10.0	9.6	7.5	5.7	3.4	1.5	63.5	1905	
Berkeley, Cal.	....	....	....	....	....	....	....	....	....	2.8	1.4	1.0	......	1904	
Berkeley, Cal.	1.0	1.4	2.1	3.1	4.7	5.7	5.5	5.1	4.6	4.3	2.7	1.3	41.6	1905	
Lake Tahoe, Cal. (elev. 6200 ft.)	**1.8**	**1.8**	**1.8**	**2.0**	**3.0**	4.2	6.2	7.1	6.2	3.6	2.6	**2.0**	42.2	1909–10	D = 4, 2 ft. high
Lake Tahoe, Cal. (elev. 6200 ft.)	**1.8**	**1.8**	**1.8**	**2.2**	**3.5**	5.0	6.5	7.4	7.8	4.3	**3.1**	**2.2**	47.4	1909–10	D = 3, 10 ft. high
Lake Tahoe, Cal.	1.1	0.9	1.2	1.8	2.4	3.2	4.4	4.9	3.8	3.1	2.2	1.3	30.5	1900–06	With some deductions E. N., Feb. 29, 1912.
Salt River, Granite Reef, near Phoenix, Ariz.	4.6	**4.8**	**6.2**	**9.0**	**11.5**	**13.5**	**14.2**	14.2	13.8	11.3	7.4	4.6	115.2	1909–10	D = 4, ground
Salt River, Granite Reef, near Phoenix, Ariz.	5.3	**5.5**	**7.2**	**10.5**	**13.5**	**15.5**	16.6	16.4	16.7	14.2	9.8	5.6	136.8	1909–10	D = 3, 10 ft. high
Salt River, Granite Reef, near Phoenix, Ariz.	**4.2**	**4.4**	**5.2**	**7.0**	**9.5**	**12.0**	12.8	12.5	11.0	8.3	6.6	4.2	97.7	1909–10	D = 4, on raft
Holbrook, Ariz.	....	....	....	....	....	....	....	6.6	4.6	4.3	1.4	0.2	......	1905	Lat. 34.5° N., long., 110° W., elev., 5072 ft., annual rainfall, 8.99 in.
Holbrook, Ariz.	....	1.2	3.3	5.1	6.9	8.6	7.4	6.2	5.1	3.4	....	....	......	1906	
Holbrook, Ariz.	....	1.2	2.3	3.8	4.0	3.9	6.4	6.4	4.6	4.0	2.5	2.4	48.6	1907	
Holbrook, Ariz.	3.4	3.4	2.8	2.8	4.9	5.2	6.8	5.2	4.1	3.5	3.2	3.3	45.4	1908	
Holbrook, Ariz.	4.1	3.3	2.0	2.8	4.3	6.8	7.8	4.1	3.6	3.1	2.1	1.4	......	1909	

* *E. N.*, June 16, 1910, unless otherwise credited. *D* diam. of test pan, ft. See also "Evaporation on United States Reclamation Projects," by Houk, *Proc. A. S. C. E.*, Jan., 1926, p. 41.

Continued on next page.

Table 13. Evaporation from Water Surfaces in Inches—(*Continued*)

Locality (also position of test pan if known)	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	12 Mos.	Year	Remarks, D = diam. of test pan, ft.
Rio Grande, Elephant Butte, N. Mex.	**2.5**	**2.8**	**4.5**	**8.0**	**11.5**	13.4	11.6	10.5	8.6	6.8	3.9	**3.0**	87.0	1909–10	D = 4 ft., ground
Rio Grande, Elephant Butte, N. Mex.	**2.8**	**3.0**	**5.2**	**10.2**	14.8	16.5	13.7	12.3	10.3	8.3	4.9	**3.2**	105.3	1909–10	D = 3, 10 ft. high
Carlsbad, N. Mex.	**5.0**	**5.5**	8.9	11.7	12.9	12.4	12.0	11.0	9.8	7.6	5.5	**5.0**	107.2	1909–10	D = 4, ground
Carlsbad, N. Mex.	**6.0**	**7.0**	9.9	12.4	13.8	13.5	13.0	10.8	9.2	7.0	6.4	**6.0**	115.0	1909–10	D = 3, 10 ft. high
Carlsbad, Alfalfa field	**5.0**	**5.2**	**9.0**	11.1	11.0	9.1	10.6	9.3	7.8	5.9	5.4	**5.0**	94.4	1909–10	D = 4, ground
Carlsbad, Alfalfa field	**6.5**	**7.0**	**10.5**	13.0	16.4	14.7	14.5	11.5	10.2	8.5	7.2	**6.5**	126.5	1909–10	D = 3, 10 ft. high
Pecos river, Lake Avalon	**4.5**	**4.5**	5.5	7.4	10.1	11.0	12.9	**12.0**	**9.5**	**7.0**	**5.8**	**4.5**	94.5	1909–10	D = 4 ft., floating
Pecos river, Lake Avalon	**6.5**	**7.0**	8.7	13.8	13.7	18.4	17.8	15.8	12.3	**9.5**	**7.5**	**6.8**	137.8	1909–10	D = 3, 10 ft. high
Truckee-Carson, Fallon, Nev.	**1.8**	**1.8**	**2.2**	**3.2**	**5.2**	7.9	9.9	8.7	5.1	3.4	**2.5**	**2.0**	53.6	1909–10	D = 4, floating
Truckee-Carson, Fallon, Nev.	**2.0**	**2.2**	**3.0**	**4.5**	**7.5**	13.0	15.8	14.2	8.7	5.3	**3.2**	**2.2**	81.9	1909–10	D = 3, 10 ft. high
Klamath, Ore., Ady	**0.5**	**1.2**	3.6	6.6	7.2	7.0	8.0	9.2	6.1	2.5	1.0	**0.5**	53.4	1909–10	D = 4, floating
Klamath, Ore., Ady	1.0	**1.8**	4.8	8.2	9.1	10.0	11.1	12.4	7.0	4.7	2.8	**1.2**	74.1	1909–10	D = 3, 10 ft. high
Klamath, Ore., Ady	1.0	**2.0**	5.0	8.8	9.8	11.3	11.9	13.3	7.9	5.0	2.8	**1.5**	80.4	1909–10	D = 3, 20 ft. high
Hermiston, Ore., Umatilla project	**1.2**	**1.2**	**3.0**	7.3	7.9	9.5	12.0	11.1	7.4	3.9	**2.0**	**1.5**	68.0	1909–10	D = 4, raft
Hermiston, Ore., Umatilla project	1.5	**1.8**	**4.2**	**9.3**	11.4	13.8	17.5	16.9	10.1	6.1	**3.0**	**1.8**	97.3	1909–10	D = 3, ground
Hermiston, Ore., Umatilla project	**2.0**	**2.2**	**5.5**	9.4	10.9	13.5	17.3	16.0	10.6	7.3	**4.0**	**2.5**	101.1	1909–10	D = 3, 25 to 65 ft.
Yakima River, Lake Kechess, Wash	**0.5**	**0.5**	**1.2**	2.6	3.8	5.5	5.9	5.5	4.4	1.5	**0.8**	**0.5**	32.8	1909–10	D = 3, 10 ft. high
N. Yakima, Wash., U. S. Reclamation project	**1.8**	**2.5**	**6.2**	7.9	8.4	8.9	10.7	9.4	5.5	3.2	**2.0**	**1.5**	68.0	1909–10	D = 4, ground
N. Yakima, Wash., U. S. Reclamation project	**2.0**	**4.2**	**8.2**	9.9	10.2	11.1	12.5	11.6	7.8	4.0	**2.5**	**2.0**	86.1	1909–10	D = 3, 10 ft. high
Snake river, Idaho, Minidoka Dam	**2.2**	**2.5**	**4.0**	7.0	11.2	12.3	15.0	**13.5**	**11.0**	**8.5**	**5.8**	**3.5**	96.5	1909–10	D = 3, 10 ft. high
Payette-Boise, Idaho, Deer Flat	**1.5**	**2.2**	**4.0**	**7.2**	10.7	11.0	11.2	11.8	**9.8**	5.4	2.7	**1.5**	79.0	1909–10	D = 3, ground
Payette-Boise, Idaho, Deer Flat	**1.8**	**2.5**	**5.5**	**9.5**	11.8	12.0	11.7	11.5	**10.5**	5.0	4.2	**1.8**	87.8	1909–10	D = 3, 20 ft. high
Payette-Boise, Idaho, Deer Flat	**2.0**	**2.8**	**4.2**	**6.0**	7.9	9.6	10.6	12.2	**9.2**	5.4	5.5	**2.0**	77.4	1909–10	D = 4, floating
Payette-Boise, Idaho, Deer Flat	**2.0**	**3.0**	**4.5**	7.0	9.9	10.8	12.5	12.5	**10.5**	5.8	3.6	**2.0**	84.1	1909–10	D = 4, 20 ft. high
Nebraska Interstate Canal, Dutch Flats	**1.8**	**1.8**	**3.0**	**4.5**	**6.2**	8.0	11.0	9.4	7.4	5.6	**4.0**	**3.0**	65.7	1909–10	D = 4, ground
Nebraska Interstate Canal, Dutch Flats	**2.0**	**2.2**	**3.5**	**6.0**	**8.5**	11.0	14.7	12.7	10.0	7.6	**5.2**	**3.0**	86.4	1909–10	D = 3, 10 ft. high
California, O., Cincinnati Filter	1.0	**1.5**	**2.5**	4.1	5.1	6.2	7.2	7.3	5.6	**3.0**	**1.5**	1.0	46.0	1909–10	D = 4, floating
California, O., Cincinnati Filter	**1.2**	**2.0**	**3.5**	6.6	7.7	8.3	9.7	8.9	6.7	**4.0**	**2.0**	**1.2**	61.8	1909–10	D = 3, 10 ft. high
Birmingham, Ala., East Lake res.	**1.5**	**1.5**	**2.2**	5.4	6.4	7.5	7.0	7.3	5.6	4.0	**2.2**	**1.5**	52.1	1909–10	D = 4, floating
Birmingham, Ala., East Lake res.	**1.5**	**1.8**	**2.5**	6.6	7.6	8.7	9.6	10.4	7.1	**5.0**	**2.8**	**1.8**	65.2	1909–10	D = 2, 10 ft. high
Birmingham, Ala., East Lake res.	**1.5**	**1.8**	**2.5**	6.9	7.7	8.9	9.8	10.9	8.9	**5.0**	**2.8**	**1.8**	66.2	1909–10	D = 2, 20 ft. high
Birmingham, Ala., East Lake res.	**1.5**	**1.8**	**2.5**	7.1	7.8	9.0	10.2	11.0	8.9	**5.0**	**2.8**	**1.8**	69.4	1909–10	D = 2, 40 ft. high
Boston, Mass., Chestnut Hill	1.0	1.0	1.7	3.0	4.5	5.5	6.0	5.5	4.1	3.2	2.2	1.5	39.2	1875–90	T.A.S.C.E., Vol. 27, 1892.
Mt. Hope Res., Rochester	0.5	0.5	1.3	2.6	3.9	4.9	5.5	5.3	4.2	3.2	1.4	1.1	34.5	1891–98	Turneaure & Russell. Public Water Supplies, 1908, p. 56.
Lea Bridge, England	0.8	0.6	1.1	2.1	**2.8**	3.1	3.4	2.8	1.6	1.1	6.7	0.6	20.6	1860–73	

Figures in heavy face type were obtained by interpolation (Abstract of Data #4, U. S. Dept. of Agr. Weath. Bur.). Observed evaporations, no correction made for wind, temperature, vapor pressure, or size of pan.

Lowcock's Formula. See *Surveyor*, Dec. 24, 1909, p. 742.

Grunsky[7a] develops a formula for calculating evaporation in terms of temperature for various altitudes:

$$E = E' + 0.0000033\sqrt{ta}$$

in which E = mean monthly rate of evaporation in ft. per 24 hr.
E' = mean monthly rate of evaporation at sea level.
t = mean monthly temperature, °F.
a = elevation in ft. above sea level.

RECORDS AND THEIR USE

Pan Tests. The U. S. Dept. of Agriculture has several bureaus engaged in making pan tests.* Observations at Reno, Nev., Aug. 1 to Sept. 15, 1907, show that the locations of the pans relative to the water of the reservoir are important in measuring total evaporation. Readings on pans distant from a reservoir cannot be transferred to the water surface without the utmost caution. See also discussion on Lake Conchos tests in *T. A. S. C. E.*, Vol. 80, 1916, p. 1851. Horton emphasizes need for experimental determination of ratio of evaporation from standard pans to evaporation from a larger surface.†,[8] From Salton sea tests, Prof. F. H. Bigelow deduced that evaporation depth from reservoir surface is 62 per cent. of that from pan. At Lake Conchos, Mexico (67.7 sq. mi. in area; average depth, 61 ft.; 4300 ft. above sea level; mean yearly temperature, 67°), Duryea and Haehl checked this ratio, with pan 3 ft. square.[6c] From pan tests, U. S. Weather Bureau concludes that if evaporation from a large water surface is 1, that from a pan of 2 ft. diam. is at rate of 1.75; diam. 4.5 ft., rate 1.50; diam. 6 ft., rate 1.30.

Variation in United States. Evaporation in the United States varies from 18 to over 100 in., annually, being greatest in the arid regions, and least at high altitudes in the north.

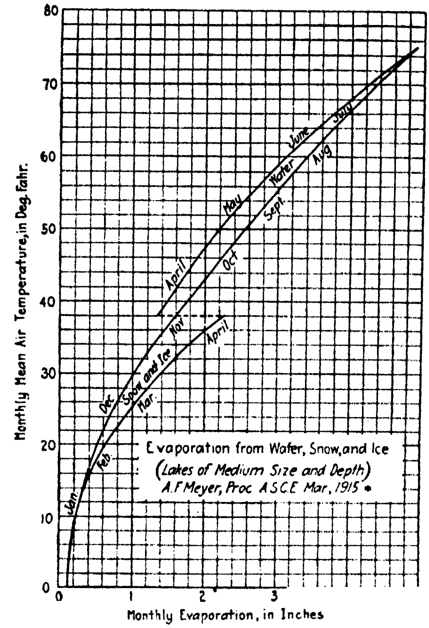
Table 14. Records of Reservoir and Land Pans at Charleston Reservoir, 19 Years, 1905–1923,[11] in Inches

	Reservoir	Land
January...	2.22	4.48
February..	2.45	4.85
March....	3.67	7.39
April......	4.71	8.72
May......	5.45	10.12
June......	5.66	10.42
July......	5.03	9.53
August....	4.54	9.02
September	4.38	7.93
October...	4.02	6.82
November	2.81	5.15
December.	2.10	4.10
Yearly....	45.83	88.54

* See Table 13, also *J. of Agr. Research*, Vol. 10, No. 5, 1917, and *Monthly Weather Review*, December, 1916.

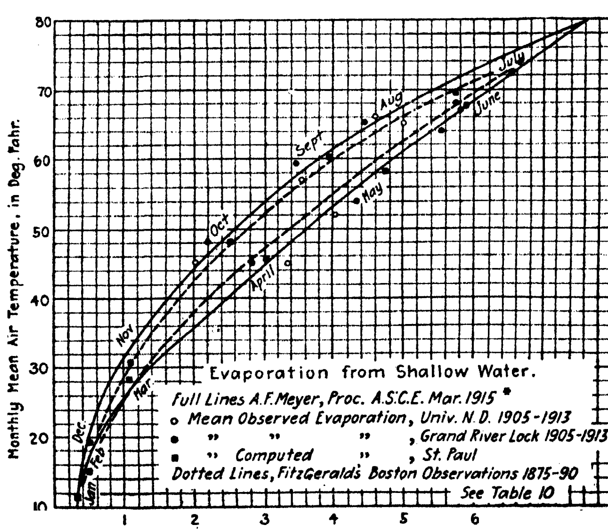
† See records in *Monthly Weather Review*, notably December, 1916, p. 674; October, 1921, pp. 553–566.

In Western United States, a committee [7b] reported to Pacific Coast Electric Light and Power Assn., (1) that in Western United States, evaporation



* T. A. S. C. E., Vol. 79, 1915, p. 1079.

FIG. 6.



* T. A. S. C. E., Vol. 79, 1915, p. 1072.

FIG. 7.

from a water surface or floating pan is two-thirds that from a land-exposed, buried pan; (2) that floating pan observations, if properly made, and if certain precautions are taken, are the most reliable for reservoir calculations.

New York State Barge Canal, in Kuichling's report (1896), is estimated to lose daily 0.30 in. by evaporation. He adds 10 per cent. to this, as a provision for consumption by aquatic plants, giving a total of 0.33 in.

Effect on Reservoir Design. Evaporation is subject to great variation due to difference in mean depth, temperature, winds, and relative humidity. Evaporation losses must be considered in all computations of reservoir capacity. They may reduce reservoir capacity 50 per cent. per annum. Careful measurements in a floating pan at Sweetwater dam, Cal., showed an annual loss of 54 in.; 2 in. in January, and 8 in. per month for July and August. This amounts to an annual loss of 15 per cent. of the stored water, and as the reservoir must hold 2 years' supply to tide over dry years, evaporation amounts to 30 per cent. of the water impounded. This leaves 70 per cent. as available capacity. At Cuyamaca reservoir on an adjacent watershed, average annual evaporation loss for 9 years was 56.7 in. This amounts to a 26 per cent. yearly loss, considerably larger than for Sweetwater, probably due to much greater exposed surface per volume stored. These instances show the advantages of deep reservoirs and high dams for effective conservation of water.[9]

Hazen's rules[10] for yield* studies; (*a*) Where average rainfall exceeds evaporation (1) allow 1 ft. at the flow line as generally sufficient. (This is arbitrary, even for Eastern States); or (2) allow a quantity calculated by deducting two-thirds of mean annual rainfall from estimated mean annual evaporation from water surface. (*b*) Where average rainfall does not exceed evaporation, compute yield for full-capacity reservoir, and deduct the net loss by evaporation, computed as a draft.

SOIL EVAPORATION

Rate of evaporation from soils depends upon: (1) Amount of precipitation; (2) moisture in the surface soil; (3) temperature of the air; (4) force of the wind; (5) proximity of the surface of saturation to the surface of the ground; (6) the slope and smoothness of the ground. The most complete observations are those made by Gilbert and Laws, in England, from 1870 to 1890, from laboratory experiments on somewhat heavy soils, uncropped, made in tanks wherein the sums of the percolation and evaporation are equal to the rainfall. For Owens Valley data, see *Water Supply and Irrigation Paper* 294, 1912.

Lysimeter Experiments, Umatilla Field Station, Hermiston, Oregon.† The lysimeters are square chambers constructed of oil-mixed concrete. They are 3.3 ft. square (inside dimension) and 6 ft. deep; equipped with a recording device below to measure the percolating water. They were constructed in 1915 and observations were begun on May 22 of that year and have been continued since that time.

One lysimeter containing a sandy soil has been kept free from plant growth to observe the quantity lost by evaporation from the soil. This quantity has been determined by subtracting from the quantity applied, both as irrigation and rainfall, the quantity collected from below the 6-ft. depth as percolate.

* See p. 89, Chap. 6.
† Information from U. S. Dept. Agri., Bureau of Plant Industry.

The difference is taken as the loss by evaporation. This loss by evaporation from the soil, which is irrigated at frequent intervals, may be compared with the loss from a free water surface exposed to similar climatic conditions.

Table 15. Evaporation from Soil in an Uncropped Lysimeter, and from a Free Water Surface in an Evaporation Tank, Both Located on a Level with the Ground Surface, Umatilla Field Station, Hermiston, Ore.

Year	Loss from soil, in.	Loss from evaporation tank, in.
1915	12.71 (after May 22)	34.28 (June–November, inclusive)
1916	12.31	38.74 (March–October, inclusive)
1917	19.86	39.97 (April–October, inclusive)
1918	22.16	40.95 (March–October, inclusive)
1919	24.55	43.15 (March–October, inclusive)

Table 16. Relation of Soil Evaporation and Percolation to Rainfall[12]

Month	Average monthly rainfall 1870–90, in.	Evaporation			Percolation at 60-in. depth		
		Average, in.	Per cent. of yearly total	Per cent. of monthly rainfall	Average, in.	Per cent. of yearly total	Per cent. of monthly rainfall
Jan......	2.51	0.45	2.7	17.9	2.06	15.2	82.1
Feb......	2.04	0.60	3.6	29.4	1.44	10.4	70.6
Mar......	1.74	0.88	5.3	50.6	0.86	6.3	49.4
Apr......	2.21	1.53	9.2	69.2	0.68	5.0	30.8
May.....	2.28	1.69	10.1	74.2	0.59	4.3	25.8
June.....	2.52	1.92	11.5	76.2	0.60	4.4	23.8
July......	3.03	2.26	13.6	74.6	0.77	5.7	25.4
Aug......	2.45	1.95	11.6	79.6	0.50	3.7	20.4
Sept.....	2.86	2.11	12.6	73.8	0.75	5.5	26.2
Oct......	3.20	1.70	10.2	53.1	1.50	11.1	46.9
Nov......	3.03	0.98	5.9	32.3	2.05	15.1	67.7
Dec......	2.42	0.61	3.7	25.2	1.81	13.3	74.8
Year	30.29	16.68	100.0	55.1	13.61	100.0	44.9

TRANSPIRATION

Transpiration denotes the water which escapes as vapor from the stomata of leaves, and the process by which such loss of moisture takes place. "Hygroscopic water," that is, the water retained in the vegetable substance produced, is inconsequential.

Transpiration Curve. Base values for total transpiration, in inches of depth, during the growing season on any given watershed, are selected with reference to character of vegetation and length of growing season, giving consideration also to available sunshine. A normal seasonal transpiration of about 9 in. has been assumed for small grains, grasses, and other agricultural crops, 8 in. for deciduous trees, 4 in. for evergreen trees, and 6 in. for small trees and brush. Normal monthly distribution of this total seasonal transpiration is based mainly on temperature. To obtain actual transpiration in any given month, values from the transpiration curve, after being multiplied by a coefficient, must be further modified on the basis of available moisture. Where precipitation minus evaporation for a given month is insufficient to meet normal plant requirements, the ground water is drawn on to a varying

extent, depending on character of root system, depth and character of soil, and quantity of surface soil storage, as determined by precipitation minus losses for previous months.[1f]

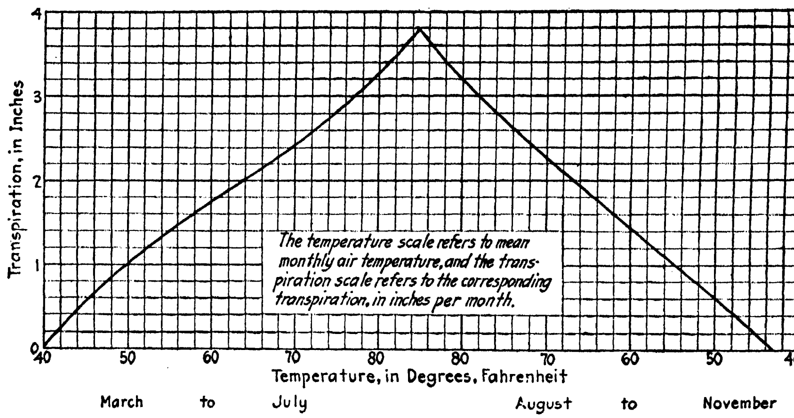


FIG. 8.—Base curve of transpiration.[1e]

Table 17. Examples of Results from Transpiration Curve (Fig. 8)

Name of watershed	Year	Total seasonal transpiration, off curve	Name of watershed	Year	Total seasonal transpiration, off curve
Little Fork........	1909	8.1	Ottertail.........	1909	9.3
Little Fork........	1910	8.5	St. Croix.........	1907	7.5
Minnesota........	1909	10.3	St. Croix.........	1912	9.8
Minnesota........	1910	12.3	Tombigbee.......	1906	20.1
Ottertail..........	1908	9.8			

In a general way, using mean monthly temperatures for watersheds in Minnesota, curve will give a normal seasonal transpiration of about 10 in. The curve takes into consideration only one factor, temperature. Character and density of vegetation, hours of sunshine, available moisture, etc., all enter in determining transpiration.

Table 18. Evaporation (Transpiration) from Various Kinds of Vegetation
Harrington

Vegetation	Proportion of evaporation from free water surface	Proportion of precipitation*
Sod........................	1.92	0.96
Cereals....................	1.73	0.86
Forest.....................	1.51	0.75
Mixed	1.44	0.72
Bare soil..................	0.60	0.30

* Warm season, May to Sept.

Bibliography, Chapter 2, Evaporation

1. A. F. Meyer: *T. A. S. C. E.*, Vol. 79, 1915. **a**, p. 1062; **b**, p. 1067; **c**, p. 1074; **d**, p. 1099; **e**, p. 1094; **f**, p. 1101. **2.** *T. A. S. C. E.*, Vol. 15, 1886, p. 589. **3.** *T. A. S. C E.*, Vol. 27, 1892, p. 275. **4.** König: "Wasserleitungen und Wasserwerke," 1907. **a**, p. 228; **b**, p. 17. **5.** *Monthly Weather Review*, February, 1908; p. 34. **6.** *T. A. S. C. E.*, Vol. 80, 1916. **a**, p. 1851; **b**, p. 1983; **c**, p. 1831. **7.** *J. Elec.* 1921. **a**, p. 25; **b**, p. 489. **8.** *E. N. R.*, Mar. 27, 1919, p. 615. **9.** Schuyler: "Reservoirs for Irrigation, Water Power and Domestic Water Supply," (Wiley, 1908) p. 237, **10.** "American Civil Engineers Handbook," 4th ed., (Wiley, 1920), p. 1195. **11.** 1923 *Report* Water Dept., *Yearbook*, City of Charleston, p. 97. **12.** *Proc. Inst. C. E.*, Vol. 105, 1891, p. 36.

CHAPTER 3

RUN-OFF AND STREAM-FLOW*

Source. The flow of a stream is derived from precipitation on the catchment area. Flood flows are direct from rainfall or melting snow, while dry-weather flow is maintained by pondage or by ground storage.

Definitions.[1a] The *run-off* of a given area is the quantity of water discharged from that area. *Yield* (Chap. 6) is the collectible portion of the surface and ground waters. *Stream-flow* is the visible portion of the run-off; it includes not only the water entering the stream over the earth's surface, but also water which is temporarily absorbed on the catchment area and tardily discharged into stream. *Watershed*, or drainage area,† of a stream, at a given point, is the area which contributes its run-off to the stream above that point.

Factors that modify or control run-off[2] are numerous and vary widely in different regions. A number are given below. Of some the effects are indicated; of others they will be obvious after a little thought. Some factors mentioned do not apply to small watersheds. On very large watersheds, there may be some counterbalancing of effects.

1. *Precipitation.* (*a*) Rain or snow.‡ (*b*) Amount of each, and the total annual precipitation. (*c*) Distribution throughout the year. (*d*) Intensity or manner of occurrence. (*e*) The character, direction, extent and duration of storms.

2. *Temperature.* (*a*) Variations on the watershed. (*b*) Relation of extreme temperatures to the occurrence of precipitation. (*c*) Accumulation of snow and ice caused by low temperature. (*d*) Occurrence of low temperature causing the freezing of the ground at times of heavy rains, resulting in excessive run-off.

3. *Topography.* (*a*) Level, or degree of inclination. (*b*) Character of the area, whether smooth or rough. (*c*) Elevation. Hazen reduces annual run-off 0.64 in. for each 100 ft. higher altitude.[4]

4. *Geology.* (*a*) Pervious or impervious. (*b*) If pervious, whether such pervious deposits are: (1) shallow or deep; (2) level or inclined; and whether the outlet or point of discharge of the pervious deposits are: (3) in the lower valley of the same river, or (4) in valleys of other rivers or in the sea. (*c*) Condition of the channel of stream, whether: (1) pervious or impervious; (2) whether the bed contains more or less extensive deposits of sand and gravel, permitting development of more or less extensive underflow. A watershed covered with loose gravel and sand will generally show a greater yield than one with a clay cover, as the rainfall sinks into the porous material and is

* For more extended discussion, see "Elements of Hydrology," by A. F. Meyer[6] (Wiley, 1917); "Hydrology," by D. W. Mead (McGraw-Hill Book Company, Inc., 1919); Report of Committee on Run-off, 1922, *Boston Soc. C. E.*; and *Technical Reports*, Miami Conservancy District, particularly Part VIII, 1921.

† "Catchment area," used in British practice, is a preferable term.

‡ Studies by *Nevada Cooperative Snow Survey* disprove the theory that dense snow assures retarded run-off.[3]

largely protected against evaporation until it drains into the streams. (*d*) Degree of saturation of soil. (*e*) Soil temperature.

5. *Condition of the Surface.* (*a*) Extent of vegetation. (*b*) Extent of cultivated areas. (*c*) Nature of vegetation—whether grass land, crops, or forests.

6. *Natural Storage.* (*a*) Nature and extent of surface storage—lakes, ponds, marshes, and swamps. (*b*) Nature and extent of ground storage, in gravel, sand, and other pervious deposits.

7. *Geography.* (*a*) Size. (*b*) Shape—long and narrow, or short and broad.* (*c*) Location relative to prevailing winds. (*d*) Direction relative to path of storms. (*e*) Relation to mountains. (*f*) Distance from the ocean or other large body of water. (*g*) Snow-capped mountains or glaciers, or high or wooded areas retaining snow late into the warm season.

8. *Character of the Stream and its Tributaries.* (*a*) Slope or gradient. (*b*) Falls and rapids. (*c*) Cross-section of the stream—deep or shallow. (*d*) Arrangement of tributaries—joining the main stream at various points along its course or concentrated in a fan-like arrangement at a common point of discharge. Length of stream channels per unit of area.

9. *Artificial Control of the Stream.* (*a*) Dams and storage reservoirs. (*b*) Restrictions by dikes and levees. (*c*) Obstruction by piers, abutments, and other encroachments in the waterway. (*d*) Diversion.

10. *Artificial Use of the Stream.* (*a*) Irrigation. (*b*) Water supply. (*c*) Supply of navigation canals. (*d*) Artificial storage and regulation.

11. *Character and Extent of the Winds.* (*a*) Intensity and direction. (*b*) Modification by mountains and forests. (*c*) Frequency and duration.

12. *Ice Formation.* (*a*) Modifying the winter flows of the stream. (*b*) Gorges and accompanying floods.

13. *Evaporation.*† (*a*) Governed largely by conditions mentioned above. (*b*) Proportion of water surface to total area.

Units. Run-off is expressed as (1) second feet per square mile of area drained; (2) depth in in. per month (or other convenient time interval); (3) per cent. of rainfall. This last has been discarded by Meyer and other recent investigators, who consider that run-off is a residual, *i.e.*, precipitation minus losses in evaporation, transpiration, interception by vegetation, and deep seepage. (4) Ratio to arithmetic mean; employed by Hall[7] in studying variations of California streams.

Importance of Data. Stream-flow data are essential in flood studies and spillway designs, while yield data are required for all water-supply investigations.

STREAM GAGINGS‡

Requisites. The object of stream gagings is to secure records from which can be deduced the flow of the stream over a period of years. It is essential that gaging stations be chosen at sites where the regimen of the stream is fairly established. A thorough knowledge of the physical characteristics of a stream

* Hearn[5] concludes that watersheds shaped to provide greatest concentration, *e.g.* semicircular areas, may have 2.5 times the run-off of sheds of triangular shape.
† See Chapter 2.
‡ See Hoyt and Grover, "River Discharge," (Wiley, 1923); and references in footnote on page 38 for particulars; see also p. 792.

is necessary to select gaging stations properly. A good station for higher flows is often useless for the smaller flows, while a station well adapted to medium flows has proved worthless for the extreme stages. Rarely can a station be used for all stream stages with equally good results. Measurements of velocity should be made at various gage heights, and determination by soundings of the waterway corresponding to these gage heights, whence discharges may be calculated. A rating table is constructed, to show discharges at various gage heights. Daily, or at times more frequent, readings of gage heights are needed. Independent discharge measurements, as a rule, are of little value unless taken at stages known to be either extremely low or extremely high. In ordinary work it is necessary to make a series of measurements which, with daily gage heights, make possible computation of total flow and its distribution.

Section.[8a] The ideal measuring section should be perpendicular to thread of current, and where conditions of bank and bed, above and below, are permanent. This section is divided into partial areas by perpendiculars terminating in surface at points where observations of depth and mean velocity in the vertical are made. The measuring points should be spaced to show any irregularities in cross-section or velocity. For repeated measurements, points should be permanently marked.

Soundings. In sounding with a rod, take care it does not sink into bed of stream, and that reading is not too high on account of water running up on it. Soundings should be made by a weight attached to a line, the weight having a pointed end upstream to offer least resistance to the current.

Velocity by Floats. Velocities of streams may be obtained for rough approximations of discharge by placing in midstream floating bodies of about the specific gravity of water, and noting the time taken to traverse a measured distance. It must be borne in mind that the velocity is greatest at the surface of the water in midstream. Experiments show that the mean velocity for a cross-section is about 83 per cent. of this maximum. A fair allowance must be made for local conditions altering this percentage. The cross-section may be estimated by measuring the depth at equal distances across, plotting a profile, and planimetering the plotted area; or by taking the average of these depths, and multiplying by the width.

Velocity.[10] With the aid of a current meter, six methods are available: (1) vertical velocity curve; (2) the single point method (one gaging in each vertical about 0.6 depth of the stream below the surface); (3) top and bottom method (1 ft. below top and 1 ft. above bottom) the average being used; (4) vertical integration method; (5) two-tenths-eight-tenths method; (6) sub-surface method.

(1)[8b] Measurements are usually made just beneath surface, at 0.5 ft. below surface, at each fifth to each tenth of depth, and as near the bottom as possible. These measured velocities, plotted with depths as ordinates and velocities as abscissas, define a vertical velocity-curve, which shows velocity at every point in vertical, and from which mean velocity can be determined by dividing area bounded by curve and axis of ordinates by mean depth. The following three methods are used in this determination: (*a*) Determine area with planimeter. (*b*) Divide area into sections of equal depth, usually ten; take mean of velocities

at mid-points of sections as mean velocity. (*c*) Divide area into sections of convenient depth, which will be equal, except bottom section which may have an odd depth. If bottom section is same depth as others, take mean of middle ordinates. If bottom section is odd depth, multiply its mean velocity by ratio of its depth to sum of middle ordinates of other sections, and divide by number of sections. This takes too long for ordinary use; it should be used only as a standard. In (2), for flood measurements, the meter is held about 1 ft. below the surface and a coefficient of 0.85 to 0.90 is applied to reduce the observed velocity to the mean for that vertical. In (4), the mean velocity for the vertical is determined by lowering the meter at a slow uniform speed from the surface to the bed, and then raising it to the surface. The velocity as indicated by the mean number of revolutions per second is taken as the mean velocity for that vertical. Special care has to be taken in lowering to avoid increasing the number of revolutions, by the vertical motion. This method is used to good advantage in high-water measurements, where the high velocity makes it impossible to hold the meter at any given point. It is also used under ice or where conditions are such that the point method is unreliable.

The six-tenths method (see (2)) has been superseded on the Catskill Waterworks of New York City by the two-tenths-eight-tenths method, which is theoretically and practically correct.

(5)[8c] In two-tenths-eight-tenths method, observations are taken at depths from surface of 0.2 and 0.8; mean velocity is taken as mean of velocities at these two points. Method is based on theory that vertical velocity-curve is a parabola. Experience proves this method gives more consistent results than any other except vertical-velocity-curve method.

(6)[8c] In the sub-surface method, measurement is made at from 0.5 to 1 ft. below surface, depending on depth of stream; meter is held at sufficient depth to be out of surface disturbance. When this method is used, velocity must be reduced by coefficient to obtain mean velocity; coefficient varies between 78 and 98 per cent., depending on depth and velocity of stream. The deeper the stream and the greater the velocity, the greater the coefficient. For average streams in moderate freshets use 90 per cent.; in flood, 90 to 95 per cent.; at ordinary stages, 85 to 90 per cent.

Velocity Variation in Cross-section.[10] The ratios of the average velocities, V_a, in the vertical cross-section to those at the surface, V_s, may be expressed:

$$\frac{V_a}{V_s} = 0.79 + \frac{2.80}{\frac{W}{D} + 8}.$$

This formula was tested by comparison with values determined by 43 measurements, at many places on streams widely different in physical characteristics; the ratio of width to depth $\left(\frac{W}{D}\right)$ varied from 7 to 110; the flow, or discharge, from 850 to 62,000 cu. ft. per sec. Errors by the formula were as follows, some being plus, some minus: In 5 cases, no error; 11 cases, 1 per cent.; 7 cases, 2 per cent.; 8 cases, 3 per cent.; 5 cases, 7 per cent., the maximum.

Gage-height recorders, intended to give a graphic record automatically, are made by Gurley, Stevens, Friez, and others. In hydrological studies for Seattle, apparatus was installed to record stream discharges automati-

cally.[11] Silt and debris will derange automatic apparatus, and further improvements are desirable.[30]

Diaphragm. For epitome of results, 1905–1914, and bibliography, see *Bull.* 672, Univ. of Wisconsin, Weidner, 1914.

Gaging Brooks of Small Flow. On brooks of small flow, a channel can be formed for current meter readings. Tests of accuracy showed discharge of 0.13 sec. ft. by meter and 0.135 by positive measurement (a box). Another reading gave 0.15, positive, and 0.14 metered. Also a small weir can be set.

Methods Compared.[28] Groat cites errors in results by current meters, 2 to over 8 per cent.; by weirs, 1; by diaphragm, ½ to 1; and by chemical tests, fraction of 1 per cent.

RECORDS

Published Data. The U. S. Geological Survey has for many years maintained gaging stations on important streams, and the annual records are published annually in *Water Supply and Irrigation Papers.* Many municipal reports, notably New York, Boston and Philadelphia, contain rainfall and run-off data from the watersheds used for their water supplies. The *Monthly Weather Review* also publishes daily river stages at many stations. Flow in California streams has been recorded and estimated in *Bull.* 5, Report to California Legislature of 1923, by State Engineering Department.

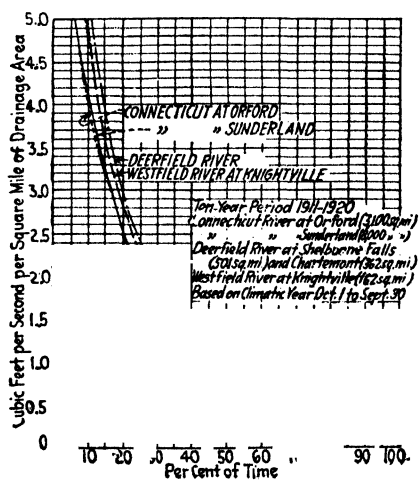


FIG. 9.—Duration curves.[1c]

(For Hudson River at Sherman Island, see T. A. S. C. E., Vol. 88, 1925, p. 1264.)

Graphical Presentations of Records. (*a*) Hydrographs are diagrams whereon time in days, weeks, months, or years is the abscissa, and flows during successive time intervals are plotted as ordinates. Maximum and minimum flows are made apparent, conveying a clearer mental picture than tabulated values; horizontal lines can be drawn to designate flood periods; peaks above a horizontal line which represents a certain chosen value, may be integrated to arrive at the volume of the flood. (*b*) Mass curves, see p. 88. (*c*) Distribution of run-off curves. (*d*) Curve of average year by months.

Distribution of Run-off Curves.[1b] First prepare a duration curve (Fig. 9), as follows: Arrange the monthly (or daily) records in order of magnitude

(reduced to cfs. per sq. mi.) from maximum down. Since records are for equal time intervals, the flows below any given magnitude may be readily calculated in terms of whole period. This furnishes the data* for plotting Fig. 9.

To prepare Fig. 10, read the intercepts of each ordinate in Fig. 9, and compute the ratio to the average flow and plot in Fig. 10. The abscissa scales are the same in Figs. 9 and 10. The Committee on Run-off of Boston Soc. C. E. reports:[1b] "This method results in a comparison of the relative distribution of run-off regardless of the size of drainage area or the disparity between average flows at the stations."

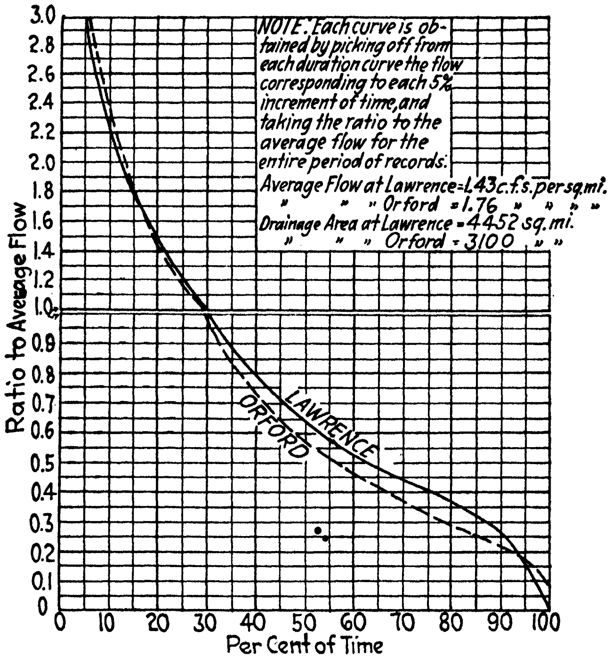


FIG. 10.—Comparison of run-off, Connecticut River at Orford, Merrimack River at Lawrence, for Period: Oct. 1, 1900–Sept. 30, 1920.[1d]

Rainfall and Run-off. *Explanation of Tables.* The following tables were compiled from *Water Supply and Irrigation Papers* on Stream Flow (U. S. Geological Survey) and from Monthly Weather Review. The second decimal is dropped from rainfall and run-off records, as explained on p. 2. Rainfall year, Oct. 1 to Sept. 30, rather than calendar year, is used. Quantities of rainfall and run-off are in inches depth. Quantities under Winter months, Summer months, and Yearly means are totals for those periods. All references to years are to rainfall year, beginning Oct. 1; *i.e.*, 1903–1906 is understood to be Oct. 1, 1903 to Sept. 30, 1906. Elevations refer to height above sea-level and are intended merely as a guide in comparing watersheds. The lower elevation represents elevation where observation was made. The upper elevation represents the source, and not the highest point on the watershed. Stations are arranged geographically.

* See also *E. N.*, Apr. 23, 1914, p. 904.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.

Connecticut River above Orford, N. H.	Time groups / Description of data	Winter months: Precipitation	Winter months: Run-off	Winter months: Percentage of run-off	Summer months: Precipitation	Summer months: Run-off	Summer months: Percentage of run-off	Yearly means, Oct. 1–Sept. 30: Precipitation	Yearly means, Oct. 1–Sept. 30: Run-off	Yearly means, Oct. 1–Sept. 30: Percentage of run-off
Descriptive		a	b		c	d		e	f	
Area, 3300 sq. mi. Steep slopes, mostly forested. 20 sq. mi. of water surface (estimate).	Means for 20 yrs., 1900–1920	11.8	12.2	97	14.5	4.8	33	34.2	23.9	70
	Max. (a)1901–02 (b)1912–13 (c)1905 (d)1917 (e)1919–20 (f)1901–02	17.0	18.5	....	19.3	8.4	....	42.4	30.2	....
	Min. (a)1904–05 (b)1903–04 (c)1913 (d)1914 (e)1910–11 (f)1910–11	8.9	7.8	....	10.5	2.6	....	30.7	17.6	....

Descriptive	Description of monthly data			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation 380–6300 ft.															
Mean temp., 1899–1920 at Chelsea, Vt. (10 mi. west of gaging station). Jan., 16°F. July., 69°F.	Monthly means	Rainfall	Amount	3.1	2.1	2.3	2.1	2.0	2.7	2.7	2.8	3.4	3.8	3.6	3.8
		Run-off	Amount	1.4	1.6	1.5	1.1	0.8	3.0	5.8	3.9	1.9	1.1	0.9	0.9
	Monthly max.	Rainfall	Year	1918	1900	1901	1909	1909	1913	1920	1912	1917	1915	1917	1907
			Amount	6.6	5.5	4.8	4.0	3.7	6.1	5.9	5.9	5.9	7.8	5.9	7.4
		Run-off	Year	1918	1918	1907	1913	1916	1903	1909	1907	1917	1915	1915, 1917	1905
			Amount	3.8	3.5	3.5	3.2	1.8	9.0	9.1	5.9	4.3	2.6	1.8	2.0
Run-off for driest 3 yrs., 1903–1906, 61.5 in. Aver., 20.5 in.	Monthly min.	Rainfall	Year	1914	1917	1914	1907	1901	1915	1911	1903	1913	1919	1913	1908
			Amount	1.1	0.7	1.0	1.6	0.7	0.4	0.9	0.3	1.8	1.7	1.7	1.0
		Run-off	Year	1908	1908	1904, 1908	1904, 1905	1905	1906	1915	1903	1914	1911	1909	1908
			Amount	0.3	0.4	0.4	0.3	0.2	0.8	3.3	1.2	0.8	0.4	0.3	0.2

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.— (*Continued*)

Sudbury River	Time groups / Description of data	Winter months, Dec.–Apr. Precipitation	Winter months, Dec.–Apr. Run-off	Winter months, Dec.–Apr. Percentage of run-off	Summer months, June–Sept. Precipitation	Summer months, June–Sept. Run-off	Summer months, June–Sept. Percentage of run-off	Yearly means, Oct. 1–Sept. 30 Precipitation	Yearly means, Oct. 1–Sept. 30 Run-off	Yearly means, Oct. 1–Sept. 30 Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 75 sq. mi. Hilly with steep slopes; large swamp areas. 30 per cent. is forested. Water surface = 5.20 sq. mi.	Means for 45 yrs., 1875–1920	19.9	14.3	70	13.7	1.9	15	44.6	20.6	46
	Max. (*a*)1890–91 (*b*)1890–91 (*c*)1889 (*d*)1889 (*e*)1890–91 (*f*)1890–91	27.9	24.8	....	20.5	6.1	....	55.9	33.4	....
	Min. (*a*)1882–83 (*b*)1910–11 (*c*)1883 (*d*)1912 (*e*)1882–83 (*f*)1910–11	12.6	6.9	....	7.3	0.1	....	27.4	8.4	....

Descriptive	Description of monthly data			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 150–500 ft.															
Mean Jan. temp., 1898–1918, 28°F. Mean July temp., 71°F.	Monthly means	Rainfall	Amount	3.7	3.8	3.8	4.0	4.2	4.3	3.6	3.3	3.2	3.6	3.8	3.4
		Run-off	Amount	0.8	1.3	1.6	2.0	2.6	4.9	3.4	1.9	0.9	0.3	0.4	0.4
	Monthly max.	Rainfall	Year	1895	1888	1901	1891	1900	1877	1904	1901	1903	1876	1898	1907
			Amount	10.7	7.2	9.7	7.0	9.1	8.4	8.9	7.2	9.2	9.1	8.2	8.8
		Run-off	Year	1890	1895	1878	1891	1886	1877	1901	1901	1903	1915	1899	1905
			Amount	3.9	4.8	5.5	5.2	8.3	8.3	7.2	5.1	3.4	1.8	2.5	2.2
Run-off for driest 3 yrs., 1908–1911, 34.2 in.; Aver. 11.4 in.	Monthly min.	Rainfall	Year	1897	1908	1875, 1877	1916	1877	1915	1892	1903	1912	1917	1883	1877, 1914
			Amount	0.5	1.0	0.9	1.5	0.7	0.1	0.8	0.9	0.5	1.1	0.7	0.3
		Run-off	Year	1909, 1912, 1914, 1916	1908	1908	1918	1901	1915	1915	1910, 1911, 1915	1914	1900, 1908, 1909, 1910, 1911, 1912, 1913	1899, 1900, 1907, 1909, 1910, 1912, 1913, 1918, 1920	1908, 1912, 1914
			Amount	0.0	0.1	0.2	0.5	0.5	1.0	1.0	0.5	0.0*	0.0*	0.0*	0.0

* Less than 0.05. See also Rainfall Records, p. 9.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Wachusett (south branch of Nashua River) at Clinton, Mass.	Time groups / Description of data	Winter months, Dec.–Apr. Precipitation	Winter Run-off	Winter Percentage of run-off	Summer months, June–Sept. Precipitation	Summer Run-off	Summer Percentage of run-off	Yearly means, Oct. 1–Sept. 30 Precipitation	Yearly Run-off	Yearly Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area,	Means for 23 yrs., 1897–1920	19.5	14.9	76	15.8	3.5	22	45.4	22.8	50
	Max. (*a*)1901–02 (*b*)1901–02 (*c*)1903 (*d*)1916 (*e*)1897–98 (*f*)1919–20	26.7	21.5	....	20.6	6.4	....	54.8	32.6	....
	Min. (*a*)1910–11 (*b*)1910–11 (*c*)1912 (*d*)1913 (*e*)1910–11 (*f*)1910–11	13.6	7.4	....	8.2	1.0	....	34.2	10.8	....

	Description of monthly data			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Mean Jan. temp., 1898–1918, 25°F. Mean July temp., 71°F.	Monthly means	Rainfall	Amount	3.2	3.5	4.1	3.6	3.9	4.1	3.8	3.4	3.8	4.1	4.1	3.8
		Run-off	Amount	0.9	1.3	2.1	2.1	2.2	4.8	3.7	2.2	1.4	0.8	0.7	0.6
	Monthly max.	Rainfall	Year	1898	1897	1901	1898	1900	1899	1901	1901	1903	1915	1898	1907
			Amount	7.2	7.6	9.4	6.6	8.7	6.8	9.6	7.0	10.4	8.6	10.6	9.5
		Run-off	Year	1898	1907	1901	1915	1900	1920	1901	1901	1903	1916	1915	1905
			Amount	2.6	4.4	5.6	5.6	7.0	8.1	8.6	4.7	3.7	1.9	2.9	2.1
Run-off for driest 3 yrs., 1910–1913, 47.0 in. Aver., 15.7 in.	Monthly min.	Rainfall	Year	1897	1902	1899	1916	1901	1915	1915, 1917	1905	1912	1917	1907	1914
			Amount	0.9	0.9	2.0	1.6	1.1	0.1	1.8	0.8	0.5	1.2	1.3	0.2
		Run-off	Year	1910	1908	1899, 1914	1918	1901	1915	1915	1905, 1911, 1915	1915	1910, 1913	1913	1917
			Amount	0.1	0.2	0.6	0.8	0.6	1.0	1.6	0.8	0.4	0.1	0.1	0.1

See also p. 10.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Croton River at New Croton Dam, New York	Time groups / Description of data	Winter months, Dec.–Apr. Precipitation *a*	Run-off *b*	Percentage of run-off	Summer months, June–Sept. Precipitation *c*	Run-off *d*	Percentage of run-off	Yearly means, Oct. 1–Sept. 30 Precipitation *e*	Run-off *f*	Percentage of run-off
Descriptive										
Area, 375 sq. mi.* Rolling, largely agricultural and grazing; water surface = 5.3 per cent. total watershed.	Means for 52 yrs., 1868–1920	20.0	15.1	76	17.4	3.3	19	48.4	23.0	47
	Max. (*a*)1908–09 (*b*)1901–02 (*c*)1887 (*d*)1901 (*e*)1900–01 (*f*)1902–03	26.9	24.1	....	26.4	8.9	....	61.4	34.0	....
	Min. (*a*)1871–72 (*b*)1870–71 (*c*)1917 (*d*)1880 (*e*)1916–17 (*f*)1870–71	10.9	7.9	....	10.2	0.9	....	33.4	13.7	....

Descriptive	Description of monthly data			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 200–1400 ft.															
Mean Jan. temp., 25°F. Mean July temp., 77°F. (1912)	Monthly means	Rainfall	Amount	4.0	3.7	3.9	4.1	4.1	4.2	3.7	3.9	3.6	4.7	4.9	4.2
		Run-off	Amount	1.1	1.5	2.0	2.6	2.8	4.3	3.3	1.9	1.0	0.6	0.9	0.8
	Monthly max.	Rainfall	Year	1913	1889	1901	1891	1900	1877	1901	1893	1903	1887	1898	1882
			Amount	9.6	8.5	8.8	9.1	7.7	8.1	8.2	7.9	11.3	11.2	11.5	14.6
		Run-off	Year	1903	1907	1878	1874	1915	1902	1901	1893	1903	1897	1875	1882
			Amount	4.8	5.0	7.1	8.3	6.5	9.9	7.5	5.5	3.2	2.6	5.1	3.4
Run-off for driest 3 yrs., 1908–1911, 52.5 in. Aver., 15.7 in.	Monthly min.	Rainfall	Year	1879	1902	1892	1896	1901	1915	1892	1887	1873	1873	1899	1914
			Amount	0.7	0.9	1.0	1.1	0.8	0.3	1.1	0.3	0.7	2.2	0.6	0.3
		Run-off	Year	1910	1908	1876, 1880	1871, 1875	1901	1872	1882	1896	1880	1913	1880, 1881, 1882, 1900, 1907	1881, 1910, 1917
			Amount	0.0	0.2	0.4	0.5	0.5	1.5	1.2	0.5	0.3	0.01	0.1	0.0

See also *Rainfall Records*, p. 12, and Croton flow in dry years, p. 65. *Fixed at 375 sq. mi. by New York City Dept. of Water Supply, Gas & Electricity, Sept. 4, 1916 (*E. N.*, Sept. 21, 1916, p. 576). This gives an error of about +4 per cent., in run-off records and percentages before that time.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(Continued)

Houssatonic River, above Gaylordsville, Conn.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 1020 sq. mi. Extensive cultivation; sparsely timbered hills.	Means for 9 yrs., 1900–09	13.3	15.5	85	18.4	5.5	30	47.1	26.6	56
	Max. (*a*)1908–09 (*b*)1901–02 (*c*)1903 (*d*)1903 (*e*)1907–08 (*f*)1902–03	25.2	21.6	....	23.3	10.6	....	52.9	36.9	....
	Min. (*a*)1906–07 (*b*)1906–07 (*c*)1909 (*d*)1908 (*e*)1904–05 (*f*)1908–09	13.0	9.9	....	13.0	2.8	....	39.8	19.4	....

	Description of monthly data[1] (9 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 230 to 1900 ft.															
Mean temp., 1898–1910, at Carmel, N. Y. (18 mi. southwest of gaging station). Jan., 25° F. July, 72° F.	Monthly means	Rainfall	Amount	4.0	2.5	4.1	3.3	3.3	3.8	3.9	3.7	4.5	4.5	4.7	4.7
		Run-off	Amount	1.8	1.6	2.3	2.3	1.9	4.7	4.3	2.5	2.0	1.3	1.2	1.2
	Monthly max.	Rainfall	Year	1907	1907	1901	1905	1909	1901	1909	1901	1903	1902	1904	1907
			Amount	6.8	5.5	6.8	4.9	6.6	5.2	6.2	6.2	10.4	7.2	7.3	10.6
		Run-off	Year	1907	1907	1902	1902, 1908	1903	1902	1901	1901	1903	1903	1903	1905
			Amount	4.2	5.4	4.1	3.2	3.7	8.8	6.4	4.5	4.3	2.5	2.2	2.2
Run-off for driest 3 yrs., 1904–07, 67.4 in. Aver., 22.5.	Monthly min.	Rainfall	Year	1908	1902	1900	1901	1901	1907	1903	1903	1908	1909	1907	1908
			Amount	2.1	0.9	2.7	1.6	0.8	1.3	2.2	1.1	1.7	2.0	1.8	1.4
		Run-off	Year	1900, 1908	1908	1908	1901	1901	1907	1907	1903	1905	1905, 1909	1907	1908
			Amount	0.4	0.3	0.5	1.0	0.5	2.7	2.6	1.1	0.9	0.6	0.6	0.3

[1] Station abandoned in 1914.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

West branch of Susquehanna River above Williamsport, Pa.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 5640 sq. mi. Narrow valley, high, wooded banks.	Means for 25 yrs., 1895–1920	15.8	14.0	88	15.6	3.7	24	40.7	22.2	55
	Max. (*a*)1907–08 (*b*) 1902–03 (*c*)1902, 1920 (*d*)1916 (*e*) 1915–16 (*f*)1911–12	19.7	19.8	....	19.7	8.1	....	46.1	28.8	....
	Min. (*a*)1918–19 (*b*)1918–19 (*c*)1909 (*d*)1914 (*e*)1899–1900 (*f*)1895–96	11.5	9.4	....	10.1	1.1	....	33.0	16.6	....

Descriptive	Description of monthly data (25 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 530–2000 ft. Mean temp., 1898–1920, Williamsport. Jan., 28°F. July, 73°F.	Monthly means	Rainfall	Amount	3.2	2.3	3.0	3.1	2.7	3.6	3.4	3.8	4.2	4.1	4.1	3.2
		Run-off	Amount	1.0	1.2	1.5	2.2	2.0	4.9	3.4	2.3	1.5	1.0	0.7	0.6
	Monthly max.	Rainfall	Year	1898, 1917	1897	1901	1915	1908	1898	1909	1919	1916	1902	1901, 1911	1907
			Amount	6.2	4.9	5.5	6.1	4.7	5.2	6.7	8.5	10.1	7.6	6.6	6.1
		Run-off	Year	1911	1919	1901	1913	1915	1902	1916	1919	1916	1902	1901	1911
			Amount	4.2	3.1	4.1	4.9	4.9	8.1	6.3	6.5	6.8	4.1	1.4	2.6
Run-off for driest 3 yrs., 1908–1911, 57.7 in. Aver., 19.2 in.	Monthly min.	Rainfall	Year	1901	1904, 1917	1896	1896, 1912	1905, 1906	1910	1915	1903, 1920	1912	1909	1907	1900, 1908, 1914
			Amount	0.9	0.5	1.2	1.5	1.0	0.6	1.2	1.7	1.9	1.7	1.4	1.0
		Run-off	Year	1895, 1899, 1908, 1909, 1910, 1914	1909	1904, 1908, 1909, 1917	1918	1920	1906	1915	1903	1914	1912	1910	1908, 1909
			Amount	0.2	0.1	0.3	0.2	0.5	1.6	1.6	0.6	0.4	0.2	0.1	0.1

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Susquehanna River above Wilkes-Barre, Pa.*	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
Descriptive	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
		a	*b*		*c*	*d*		*e*	*f*	
Area, 9810 sq. mi. Rolling, broken country in N. Y., mountains in Pa.	Means for 21 yrs., 1899–1920	13.6	12.7	93	15.6	2.9	18	38.0	19.1	50
	Max. (*a*)1901–03 (*b*)1901–02 (*c*)1915 (*d*)1917 (*e*)1901–02 (*f*)1902–03	16.7	20.8	....	21.3	7.2	....	44.1	27.2	....
	Min. (*a*)1916–17 (*b*)1916–17 (*c*)1908 (*d*)1913 (*e*)1908–09 (*f*)1910–11	9.1	7.4	....	9.8	0.9	....	30.4	11.9	....

Descriptive	Description of monthly data (21 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, source 1193 ft. At Wilkes-Barre 525 ft. Mean temp., 1898–1910 at Wilkes-Barre. Jan., 23°F. July, 74°F.	Monthly means	Rainfall	Amount	3.2	2.2	2.7	2.6	2.4	2.9	3.0	3.3	4.0	4.3	3.9	3.4
		Run-off	Amount	0.8	1.0	1.6	2.1	1.7	4.1	3.2	1.6	1.0	0.8	0.5	0.5
	Monthly max.	Rainfall	Year	1917	1900	1901	1915	1910	1903, 1913	1914	1901	1917	1902	1903	1912
			Amount	6.4	4.7	5.6	4.6	4.0	4.8	5.0	5.4	6.9	7.9	6.5	6.6
		Run-off	Year	1903	1907	1901	1913	1904	1902	1916	1909	1917	1902	1915	1905, 1912
			Amount	3.2	2.1	4.9	4.2	3.9	7.8	6.6	3.3	3.0	3.4	2.2	1.4
Run-off for driest 3 yrs., 1908–1911, 42.1 in. Aver., 14.0 in.	Monthly min.	Rainfall	Year	1910	1917	1919	1912, 1916	1901, 1905, 1917	1910	1915	1903	1912	1910	1907	1908
			Amount	1.2	0.8	1.5	1.4	1.2	0.8	1.4	1.1	1.5	2.0	1.2	1.3
		Run-off	Year	1899, 1900, 1909, 1910	1909	1908, 1909, 1914	1918	1920	1915	1915	1903	1915	1912	1902, 1910, 1911, 1913	1908, 1909, 1910, 1913
			Amount	0.1	0.1	0.2	0.2	0.3	1.4	1.5	0.4	0.3	0.1	0.1	0.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Susquehanna River above Harrisburg, Pa.	Time groups: Description of data	Winter months, Dec–Apr: Precipitation	Winter months, Dec–Apr: Run-off	Winter months, Dec–Apr: Percentage of run-off	Summer months, June–Sept: Precipitation	Summer months, June–Sept: Run-off	Summer months, June–Sept: Percentage of run-off	Yearly means, Oct 1–Sept 30: Precipitation	Yearly means, Oct 1–Sept 30: Run-off	Yearly means, Oct 1–Sept 30: Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area 28,030 sq mi Narrow valley, high, wooded banks in Pa, rolling broken country in N Y	Means for 29 yrs 1891–1920	14.6	12.8	88	15.5	3.4	22	39.5	20.2	51
	Max (*a*)1912–13 (*b*)1902–03 (*c*)1915 (*d*)1916 (*e*)1911–12 (*f*)1902–03	19.2	19.2		20.5	6.1		46.7	28.0	
	Min (*a*)1896–97 (*b*)1918–19 (*c*)1908 (*d*)1895, 1900 (*e*)1899, 1900 (*f*)1910–11	11.4	8.4		10.4	1.4		31.6	14.4	

Descriptive	Description of monthly data (29 yrs)			Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept
Elevation, 300–1200 ft															
Mean temp, 1898–1910 at Harrisburg Jan, 30°F July, 75°F	Monthly means	Rainfall	Amount	3.2	2.3	2.9	2.8	2.6	3.2	3.1	3.9	4.0	4.1	4.1	3.4
		Run-off	Amount	0.9	1.1	1.5	2.0	1.9	4.1	3.3	2.1	1.3	0.8	0.7	0.6
	Monthly max	Rainfall	Year	1917	1897, 1900	1901	1915	1893	1913	1912	1894	1916	1902	1903	1912
			Amount	6.3	4.4	5.6	5.1	4.6	5.7	5.2	7.7	6.6	7.2	6.5	7.3
		Run-off	Year	1903	1894	1901	1913	1915	1902	1916	1894	1916	1902	1915	1903, 1911, 1912
			Amount	2.2	2.2	3.5	4.3	4.1	7.5	6.2	4.5	4.3	3.2	1.8	1.4
Run-off for driest 3 yrs, 1908–1911, 48.9 in Aver, 16.3 in	Monthly min	Rainfall	Year	1892	1917	1896	1897, 1912, 1916, 1919	1901	1910	1896	1903	1912	1909	1907	1914
			Amount	1.0	0.7	1.0	1.8	0.9	0.7	1.3	1.3	2.0	2.2	1.6	1.2
		Run-off	Year	1895, 1899, 1900, 1908, 1909, 1914	1909, 1914	1908, 1914	1918	1905	1915	1915	1896	1895, 1899	1900, 1911	1895, 1900, 1909, 1910	1895, 1896, 1900, 1908, 1909, 1913, 1919
			Amount	0.2	0.2	0.3	0.3	0.5	1.7	1.3	0.6	0.5	0.3	0.2	0.2

* Data, 1910 to 1920 furnished by Penn Dept Forests and Waters

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

James River above Buchanan, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 2060 sq. mi. Open country.	Means for 19 yrs. 1895–1914	16.5	10.7	65	16.0	3.5	22	40.3	17.5	40
	Max. (*a*)1902–03 (*b*)1901–02 (*c*)1901 (*d*)1901 (*e*)1900–01 (*f*)1900–01	22.3	18.0	....	23.2	8.3	....	49.8	26.3	....
	Min. (*a*)1903–04 (*b*)1904–05 (*c*)1897 (*d*)1899, 1914 (*e*)1903–04 (*f*)1903–04	10.3	5.4	....	10.3	1.3	....	30.4	11.4	....

Descriptive	Description of monthly data (14 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 830–2500 ft.....															
Mean temp., 1898–1910, at Clifton Forge, Va. (20 mi. northwest of gaging station). Jan., 33° F. July, 75° F.	Monthly means	Rainfall	Amount	3.2	2.3	3.1	3.1	3.1	3.9	2.9	4.0	4.8	4.4	3.6	3.2
		Run-off	Amount	0.8	0.8	1.9	1.9	2.2	3.1	2.2	1.8	1.4	0.9	0.7	0.5
	Monthly max.	Rainfall	Year	1898	1896	1901	1903	1897	1912	1901	1901, 1905, 1912	1910	1905	1901	1896
			Amount	8.1	5.2	7.6	4.6	5.8	7.1	6.5	6.3	8.6	8.5	8.7	6.2
		Run-off	Year	1906	1900	1901	1908	1897	1899	1901	1901	1910	1905	1901	1907
			Amount	4.1	2.5	4.8	3.7	5.3	5.7	5.0	3.6	4.5	3.0	2.7	1.2
Run-off for driest 3 yrs., 1903–1906, 37.7 in. Aver. 12.6 in.	Monthly min.	Rainfall	Year	1896, 1904	1909	1896	1897	1906	1904, 1905	1896, 1897, 1902	1903	1914	1902	1910	1897
			Amount	0.5	0.6	0.3	1.8	0.4	2.4	1.6	1.2	1.9	2.3	1.6	1.1
		Run-off	Year	1895, 1897, 1899, 1904	1897, 1899, 1904, 1905, 1910	1903, 1904	1897	1905	1898, 1910	1913	1896	1914	1899	1899, 1900, 1912	1897, 1902, 1904, 1914
			Amount	0.2	0.2	0.3	0.5	0.5	1.5	0.5	0.6	0.4	0.2	0.2	0.2

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(Continued)

James River above Cartersville, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 6230 sq. mi. Open country, rolling.	Means for 16 yrs., 1898–1914	16.8	10.3	61	16.3	3.8	24	42.6	17.5	41
	Max. (*a*)1902–03 (*b*)1902–03 (*c*)1901 (*d*)1901 (*e*)1900–01 (*f*)1902–03	21.6	15.8	....	24.8	8.7	....	54.8	24.7	....
	Min. (*a*) 1903–04 (*b*) 1903–04 (*c*) 1914 (*d*) 1914 (*e*) 1903–04 (*f*) 1903–04	10.4	5.3	....	12.0	1.4	....	30.6	10.5	....

Descriptive	Description of monthly data (16 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 265–4000 ft.....															
Mean temp., 1898–1910, at Columbia, Va. (5 mi. northwest of gaging station). Jan., 37° F. July, 76° F.	Monthly means	Rainfall	Amount	3.6	2.2	3.6	3.3	2.9	3.8	3.2	3.7	4.8	4.0	4.3	3.2
		Run-off	Amount	0.9	0.8	1.4	2.0	1.9	2.8	2.1	1.6	1.2	0.8	0.8	0.6
	Monthly max.	Rainfall	Year	1898	1907	1901	1913	1903	1912	1901	1912	1903	1905	1901	1912
			Amount	8.4	5.1	7.2	5.4	5.1	7.2	6.9	7.1	7.7	7.5	10.2	6.2
		Run-off	Year	1906	1910	1901	1908	1899	1899	1901	1901	1907	1905	1901	1907
			Amount	3.8	1.0	3.3	3.1	3.7	5.3	4.4	3.4	3.4	2.5	3.1	1.3
Run-off for driest 3 yrs., 1903–1906, 39.8 in. Aver., 13.3 in.	Monthly min.	Rainfall	Year	1904	1905	1914	1907	1901	1910	1899	1911, 1914	1914	1902, 1909	1912	1914
			Amount	0.5	0.8	2.0	1.7	0.6	0.8	1.7	1.5	2.7	2.4	1.3	1.5
		Run-off	Year	1904	1904	1903, 1904, 1912	1904	1901	1910, 1911	1905	1911	1911, 1914	1911	1900, 1911, 1912, 1914	1914
			Amount	0.2	0.3	0.5	0.7	0.6	1.4	1.0	0.7	0.4	0.3	0.3	0.2

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

North (of James) River above Glasgow, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		a	b		c	d		e	f	g
Area, 830 sq. mi. Steep slopes, some swamps.	Means for 10 yrs. 1895–1905	15.9	9.6	60	15.9	3.6	23	40.8	16.0	39
	Max. (a)1902–03 (b)1901–02 (c)1896 (d)1901 (e)1900–01 (f)1900–01	21.8	16.1	....	22.4	7.7	....	51.5	21.3	....
	Min. (a)1903–04 (b)1904–05 (c)1902 (d)1899 (e)1903–04 (f)1899–1900	11.4	5.4	....	10.0	1.1	....	32.5	12.2	....

	Description of monthly data (10 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 1100–2000 ft.															
Mean temp., 1898–1910, at Lexington, Va. (8 mi. north of gaging station). Jan., 35° F. July, 73° F.	Monthly means	Rainfall	Amount	2.6	2.4	2.9	2.9	3.6	3.9	2.7	4.0	4.8	4.1	3.8	3.2
		Run-off	Amount	0.7	0.6	1.2	1.4	2.3	2.8	1.8	1.5	1.1	1.0	0.8	0.6
	Monthly max.	Rainfall	Year	1898	1896	1901	1902	1897, 1903	1899	1901	1901	1903	1896, 1905	1901	1896
			Amount	8.7	5.3	7.6	3.2	5.5	6.4	7.1	6.2	8.7	6.2	7.5	6.7
		Run-off	Year	1898	1896	1901	1903	1897	1899	1901	1897, 1901	1901	1905	1901	1896
			Amount	2.5	1.7	4.5	2.8	4.0	4.4	4.2	2.8	3.0	2.8	2.5	2.1
Run-off for driest 3 yrs., 1897–1900, 42.4 in. Aver. 14.1 in.	Monthly min.	Rainfall	Year	1896	1899	1896	1897	1901	1905	1896	1903	1898	1902	1900	1897
			Amount	0.3	0.7	0.2	1.8	0.5	2.7	1.2	1.3	2.7	2.2	1.5	0.7
		Run-off	Year	1899, 1904	1904	1899, 1904	1897	1901	1898	1905	1902	1899	1899	1902	1897, 1902, 1904
			Amount	0.2	0.2	0.3	0.6	0.4	1.3	0.8	0.6	0.3	0.2	0.2	0.2

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Roanoke River above Roanoke, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 390 sq. mi. Coastal plain, heavily timbered.	Mean for 9 yrs. 1896–1905*	16.5	9.8	59	16.8	4.4	26	42.7	17.7	41
No lakes. Swampy.	Max. (*a*)1902–03 (*b*)1898–99 (*c*)1901 (*d*)1901 (*e*)1900–01 (*f*)1900–01	22.3	19.8	....	26.6	12.1	....	58.3	29.7	....
	Min. (*a*)1903–04 (*b*)1903–04 (*c*)1902 (*d*)1902 (*e*)1901–02 (*f*)1903–04	10.0	3.2	....	10.4	1.4	....	34.5	8.9	....

Roanoke River above Roanoke, Va.	Description of monthly data (9 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 980–3000 ft.															
Mean temp., 1898–1908, at Roanoke. Jan., 36° F. July, 76° F.	Monthly mean	Rainfall	Amount	2.7	2.5	2.9	2.8	3.9	4.0	2.8	4.2	4.8	4.9	3.8	3.3
		Run-off	Amount	0.9	0.8	1.3	1.4	2.3	2.9	1.9	1.8	1.1	1.2	1.3	0.8
	Monthly max.	Rainfall	Year	1898	1902	1901	1903	1897	1899, 1903	1910	1901	1904	1905	1901	1900
			Amount	6.3	4.6	7.4	4.1	7.1	6.5	6.5	7.5	8.1	11.6	10.7	5.2
		Run-off	Year	1898	1900	1898	1899	1899	1899	1901	1901	1901	1905	1901	1903
			Amount	3.1	1.7	2.0	3.3	5.6	7.5	4.9	4.4	2.5	3.5	5.7	1.5
Run-off for driest 3 yrs., 1902–05, 44.4 in. Aver. 14.8 in.	Monthly min.	Rainfall	Year	1904	1901	1896	1897	1898	1905	1897	1903	1897	1900	1902	1904
			Amount	0.1	1.0	0.5	1.6	0.6	2.2	1.7	0.9	1.9	3.1	1.0	1.2
		Run-off	Year	1899, 1904	1897	1897, 1903, 1904	1897	1904	1898	1904	1902	1902	1899, 1902	1899, 1900, 1902	1902
			Amount	0.3	0.2	0.4	0.2	0.6	0.9	0.6	0.8	0.5	0.4	0.3	0.2

* Subsequent records are fragmentary.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Roanoke River above Randolph, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 3080 sq. mi.	Means for 5 yrs. 1900–1905*	17.5	9.5	54	17.4	5.6	32	44.0	18.7	24
	Max. (*a*) 1902–03 (*b*) 1901–02 (*c*) 1901 (*d*) 1901 (*e*) 1900–01 (*f*) 1900–01	24.6	14.0	....	24.1	10.3	....	54.0	25.2	....
	Min. (*a*) 1903–04 (*b*) 1903–04 (*c*) 1902 (*d*) 1902 (*e*) 1903–04 (*f*) 1903–04	10.2	4.6	....	11.8	3.6	....	34.0	11.0	....

	Description of monthly data (5 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, approx. 400–3000 ft.															
Mean temp., 1902–1910, at Saxe, Va. (2 mi. northeast of gaging station). Jan., 38° F. July, 76° F.	Monthly means	Rainfall	Amount	2.6	2.2	3.9	3.1	3.5	3.6	3.3	4.0	4.5	4.9	5.1	2.8
	Monthly means	Run-off	Amount	1.0	0.8	1.7	1.6	2.0	2.3	1.9	1.7	1.3	1.4	1.8	1.0
	Monthly max.	Rainfall	Year	1902	1900	1901	1903	1903	1903	1901	1905	1904	1905	1901	1901
			Amount	5.5	3.2	7.8	4.4	5.6	6.5	6.0	6.4	6.0	8.7	11.2	3.3
		Run-off	Year	1902	{1900 1901	1901	1903	1902	1903	1901	1901	1901	1905	1901	1901
			Amount	1.8	1.1	3.6	2.3	3.5	4.1	3.5	3.2	1.7	2.4	4.9	1.4
Run-off for driest 3 yrs., 1902–1905, 47.0 in. Aver. 15.7 in.	Monthly min.	Rainfall	Year	1904	1901	1903	1904	1901	1905	1904	1903	1905	1902	1902	1904
			Amount	0.6	1.2	1.8	2.2	0.9	2.3	1.5	1.9	2.8	2.7	2.4	1.9
		Run-off	Year	1904	1904	{1903 1904	1904	1901	1904	1904	1904	1905	{1902 1904	1902	1904
			Amount	0.3	0.3	0.7	0.8	1.0	1.0	0.8	1.1	1.0	0.8	0.8	0.6

* Rainfall records discontinued Aug. 12, 1906.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Shenandoah River above Millville, W. Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 3000 sq. mi. Broad valley and overflowed banks.	Means for 13 yrs., 1895–1908	14.1	7.7	55	16.4	3.4	21	39.1	14.1	36
	Max. (*a*)1901–02 (*b*)1901–02 (*c*)1896 (*d*)1901 (*e*)1900–01 (*f*)1901–02	19.1	16.0	....	22.3	7.1	....	48.1	19.8	....
	Min. (*a*)1903–04 (*b*)1903–04 (*c*)1897 (*d*)1902 (*e*)1903–04 (*f*)1903–04	8.7	3.6	....	9.7	1.6	....	30.5	7.9	....

Descriptive	Description of monthly data (13 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 230–1380 ft.....															
Mean temp., 1898–1910, at Martinsburg, W. Va. (15 mi. northwest of gaging station). Jan., 31° F. July, 75° F.	Monthly means	Rainfall	Amount	2.6	2.1	2.8	2.6	2.9	3.4	2.5	3.8	5.1	4.2	4.0	3.2
		Run-off	Amount	0.9	0.6	1.1	1.4	1.5	2.0	1.7	1.4	1.3	0.8	0.9	0.5
	Monthly max.	Rainfall	Year	1898	1896	1901	1903	1897	1899	1901	1908	1903	1896	1898	1896
			Amount	7.1	4.2	6.1	4.1	6.3	5.1	6.2	6.5	7.6	6.2	7.7	7.2
		Run-off	Year	1906	1906	1901	1908	1897	1902	1901	1901	1901	1901	1898	1907
			Amount	3.2	1.1	3.1	2.9	3.9	5.3	4.8	3.4	3.1	1.7	3.2	1.0
Run-off for driest 3 yrs., 1903–1906, 27.6 in. Aver., 9.2 in.	Monthly min.	Rainfall	Year	1896	{1899 1903	1896	1897	1901	1908	{1896 1899	1907	1899	1902	1897	1897
			Amount	0.5	0.8	0.2	1.4	0.3	1.8	1.2	1.8	2.1	2.2	1.4	1.0
		Run-off	Year	{1895 1904 1905	{1895 1904 1905	{1895 1904	{1897 1904	1901	1898	1905	1905	1902	1899	1900	{1897 1904
			Amount	0.2	0.2	0.3	0.5	0.5	0.9	0.7	0.5	0.5	0.3	0.3	0.2

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Continued*)

Potomac River above Point of Rocks, Md.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 9650 sq. mi. Steep, cultivated slopes; no lakes.	Means for 25 yrs. 1895–1920	14.8	8.6	58	15.1	2.8	18	38.0	13.7	35
	Max. (*a*)1914–15 (*b*)1901–02 (*c*)1903 (*d*)1901 (*e*)1914–15 (*f*)1901–02	24.0	18.5	....	20.3	5.4	....	51.3	21.5	....
	Min. (*a*)1903–04 (*b*) {1895–96, 1908–09} (*c*)1902 (*d*)1914 (*e*)1903–04 (*f*)1895–96	8.6	5.2	....	10.4	1.2	....	29.4	8.2	....

Descriptive	Description of monthly data (25 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 300–1200 ft.....															
Mean temp., 1896–1904, at Sharpsburg, Md. (20 mi. northwest of gaging station). Jan., 31° F. July, 76° F.	Monthly means	Rainfall	Amount	2.6	1.9	2.7	2.8	2.6	3.6	3.0	3.3	4.3	3.9	4.1	2.8
		Run-off	Amount	0.5	0.5	0.9	1.6	1.7	2.6	1.9	1.3	1.1	0.6	0.6	0.4
	Monthly max.	Rainfall	Year	1898	1897	1901	1911	1897	1912	1918	1908	1910	1905	1915	1896
			Amount	6.4	4.1	5.7	4.9	5.9	5.0	6.1	7.8	6.7	6.6	7.9	6.1
		Run-off	Year	1906	1913	1901	1915	1897	1902	{1901, 1918}	1908	1910	1903	1898	1911
			Amount	2.0	1.6	3.1	3.4	4.6	6.5	4.6	3.8	3.1	1.5	2.7	1.5
Run-off for driest 3 yrs., 1903–1906, 30.5 in. Aver., 10.2 in.	Monthly min.	Rainfall	Year	1901	{1909, 1917}	1896	1897	1901	1910	1900	1902	1898	1901	1910	1898
			Amount	0.6	0.5	0.4	1.6	0.5	0.4	1.3	1.4	1.8	1.2	1.5	1.3
		Run-off	Year	{1895, 1904}	*	1905	1897	1901	{1909, 1915}	1915	1896	{1902, 1911, 1914}	{1898, 1899, 1909, 1911, 1915}	{1900, 1909, 1910}	†
			Amount	0.1	0.2	0.1	0.5	0.4	1.0	0.5	0.3	0.4	0.3	0.2	0.2

* 1895, 1897, 1903, 1904, 1909, 1910, 1914, 1916. † 1896, 1900, 1902, 1904, 1909, 1910, 1913, 1914, 1917, 1919.

Table 19. Rainfall and Run-off of Representative Watersheds in Eastern U. S.—(*Concluded*)

Appomattox River above Mattoax, Va.	Time groups	Winter months, Dec.–Apr.			Summer months, June–Sept.			Yearly means, Oct. 1–Sept. 30		
	Description of data	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off	Precipitation	Run-off	Percentage of run-off
Descriptive		*a*	*b*		*c*	*d*		*e*	*f*	
Area, 745 sq. mi.	Means for 5 yrs., 1900–05	16.9	9.9	59	17.2	3.9	23	43.0	16.5	38
	Max. (*a*)1902–03 (*b*)1902–03 (*c*)1901 (*d*)1901 (*e*)1900–01 (*f*)1902–03	22.9	16.6	....	24.2	6.3	....	53.0	25.2	....
	Min. (*a*)1903–04 (*b*)1900–01 (*c*)1902 (*d*)1902 (*e*)1903–04 (*f*)1903–04	9.9	6.7	....	13.1	2.5	....	30.8	10.9	....

	Description of monthly data (5 yrs.)			Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Elevation, 220–730 ft.	Monthly means	Rainfall	Amount	2.7	2.2	3.8	3.3	3.1	3.6	3.1	4.0	4.0	4.1	6.3	2.9
		Run-off	Amount	0.6	0.6	1.4	1.7	2.3	2.3	2.1	1.4	0.9	0.7	1.4	0.8
Mean temp., 1898–1910, at Petersburg, Va. (30 mi. southeast of gaging station). Jan., 39° F July, 78° F.	Monthly max.	Rainfall	Year	1902	1902	1901	1903	1903	1903	1901	1901	1902	1905	1901	1901
			Amount	7.6	4.0	8.1	4.8	5.2	6.0	6.0	7.3	5.0	7.0	13.1	4.2
		Run-off	Year	1902	1902	1902	1903	1903	1903	1903	1901	1903	1903	1901	1904
			Amount	1.4	1.3	2.5	2.7	3.6	4.0	3.7	3.2	1.5	1.3	4.0	1.2
Run-off for driest 3 yrs., 1902–05, 48.0 in. Aver., 16.0 in.	Monthly min.	Rainfall	Year	1901	1901	{1903, 1904}	1904	1901	1905	1904	1902	1901	1902	1904	1903
			Amount	0.4	1.1	1.9	2.0	0.9	2.4	1.1	1.7	3.2	1.9	2.7	2.3
		Run-off	Year	{1900, 1905}	1900	1900	1904	1901	1901	1904	{1902, 1903, 1904}	1905	{1902, 1904}	1904	1902
			Amount	0.3	0.3	0.6	0.6	0.4	0.8	0.8	0.9	0.5	0.4	0.5	0.4

Table 20. Rainfall and Run-off, Southern California,* near San Diego†[33]

Year	Barrett Dam rain gage. Elev. 1700 ft.	Morena Dam rain gage. Elev. 3300 ft.	Run-off from Cottonwood water-shed at Barrett, 250 sq. mi., in million gallons	
			Total	Per sq. mi.
1906........	29.9 in.	34.7 in.	19,506	78.0
1907........	12.8 in.	18.5 in.	11,080	44.3
1908........	16.8 in.	20.5 in.	4,227	16.9
1909........	24.5 in.	33.0 in.	9,414	37.7
1910........	11.3 in.	13.9 in.	5,500	22.0

Season July 1–June 30	Rainfall at Sweetwater Dam, in.	Run-off, in million gallons from 186 sq. mi.		Season July 1–June 30	Rainfall at Sweetwater Dam, in.	Run-off, in million gallons from 186 sq. mi.	
		Total	Per sq. mi.			Total	Per sq. mi.
1887–88	– – – – –	2,302.6	12.4	1899–00	5.5	0	0
1888–89	13.5	8,250.1	44.4	1900–01	7.0	270.5	1.5
1889–90	13.5	6,707.9	36.1	1901–02	4.9	0	0
1890–91	12.6	7,045.9	37.9	1902–03	5.7	0	0
1891–92	9.9	2,024.8	10.9	1903–04	6.4	0	0
1892–93	11.6	5,312.1	28.6	1904–05	15.5	4,495.4	24.2
1893–94	6.2	437.1	2.4	1905–06	15.5	11,434.5	61.5
1894–95	16.2	23,983.7	128.9	1906–07	12.9	9,801.0	52.7
1895–96	7.3	431.2	2.3	1907–08	10.5	1,234.0	6.6
1896–97	11.0	2,251.3	12.1	1908–09	11.8	3,910.2	21.0
1897–98	7.0	1.3	0.0	1909–10	10.9	2,859.3	15.4
1898–99	5.0	80.0	0.4	1910–11	10.0	1,095.8	5.9
Total in 7 years, 1897–1898 to 1903–1904...............						351.8	1.9

* See also p. 22.

† For rainfall and run-off at the Continental Divide, see *E. N. R.*, Jan. 29, 1925, p. 190. For run-off near Continental Divide, see Horton, *E. N. R.*, Feb. 28, 1924, p. 355; and May 15, 1924, p. 871.

High Velocities in Stream Channels. The highest velocities which the Geological Survey has measured in natural channels were taken on Grand River at Glenwood Springs, Colo.; several measurements ranged from 10 to 15 ft. per sec. These measurements were not made at the maximum stage and it is probable that the velocity of this stream exceeds 20 ft. per sec. at times. As Grand River is a typical mountain stream, there are probably many other streams which have velocities as high as 20 ft. (Mean velocity of cross-section.)

Flood Flows.* For list giving flood flows in cu. ft. per sec. (averaged over 24 hr.), catchment area and duration of record, see paper by Weston E. Fuller, *T. A. S. C. E.*, Vol. 77, 1914, p. 564, and p. 650; Part IV, *Technical Reports*, Miami Conservancy District, 1918, p. 66; *T. A. S. C. E.*, Vol. 85, 1922, p. 1393 and p. 1528; Vol. 86, 1923, p. 282; Vol. 87, 1924, p. 135; *Proc. A. S. C. E.*, Dec., 1924, p. 1563; also *E. N. R.*, July 3, 1919, p. 30.

Niagara River Flow. Quantity flowing over Niagara Falls is 220,000 to 265,000 cu. ft. per sec. according to one authority. About 5 per cent. flows over the American Falls. Average flow is very uncertain, due to wind conditions on Lake Erie. According to gagings of Canadian government engineers at the time of a controversy over water to be taken for power, the flow

* Montana streams, see ref. 22; New York streams, ref. 23.

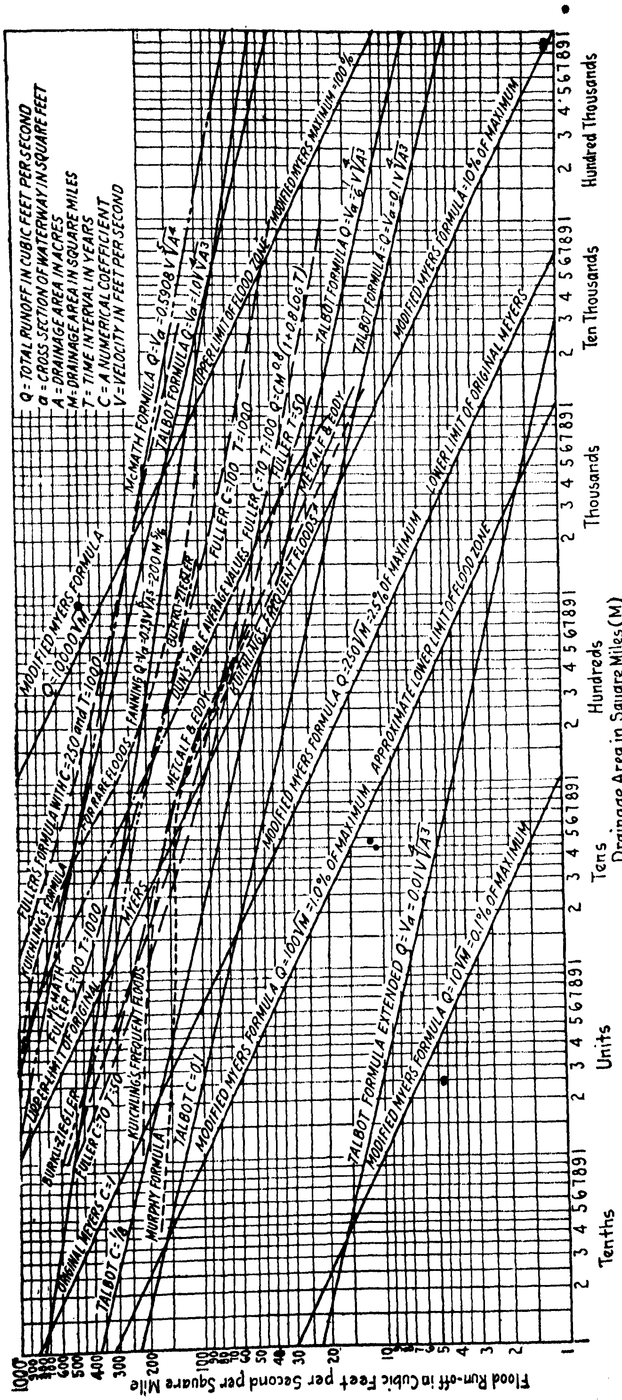


FIG. 11.*—Relation of various run-off formulas to drainage areas.[32]

* See also Fig. 23, p. 94.

varies from 300,000 to 700,000 cu. ft. per sec. Flow was estimated at 275,000 cu. ft. by John Bogart in 1890. For best information, see Shenehon's *Report* in Senate Document 105, Sixty-second Congress, 1911; and "Niagara Power,—Its Development and Distribution," by Edward Dean Adams, New York, 1925.

Honey Creek[12] above New Carlisle, Ohio. Duration of rainfall, 3 hr. Houk concluded from investigations: (1) Maximum rainfall intensity exceeded 6 in. in 3 hr. (2) Depth over 15 sq. mi. exceeded 2 in. in 3 hr. (3) Maximum rate of run-off from 6.7 sq. mi. averaged 2200 sec. ft. per sq. mi. A rainfall of 0.62 in. within 24 hr. had fairly wet the ground. All but 2 per cent. under cultivation. Glacial till, but not unusually pervious.

Unusual Stream-flow in the Southern States.* Widespread floods in the Southern States, May and June, 1901, were reported to the U. S. Geological Survey[13] by E. W. Myers. Six days subsequent to May 17, 4.49 in. of rain fell in North Carolina; the normal fall for 6 days is about 0.84 in. The maximum fall for 24 hr. for the eastern district was 3.33 in., twenty-five times the normal. Local records for 48 hr., 7.95 in. (at Marion); for 6 days, 8.68 in. (at Chapel Hill). The Catawba River, near Rock Hill, S. C., gave a flood discharge of 150,800 cu. ft. per sec. from a drainage area of 2987 sq. mi. = 50 cu. ft. per sec. per sq. mi. The greatest previous flood was less than 100,000 cu. ft. per sec. Cane Creek, Mitchell Co., N. C., has 1650 ft. fall in 11 mi. The banks are steeply sloping, covered with thin soil. On account of the great fall, the ordinary flood height rarely exceeded 6 ft.; 1901 flood rose to 12 ft. Boulders of 2 to 8 cu. yds., weighing from 4 to 16 tons, were carried 100 to 300 ft. Velocity, V, required to move bodies of mean diam., D, and specific gravity, g, immersed in water, was derived by Ganguillet and Kutter's formula, $V = 5.67\sqrt{Dg}$. If $g = 2.7$ (gneiss), and $D = 5$ ft., $V = 20.6$ ft. per sec. The area of watershed was 22 sq. mi.; the extreme low water discharge, 8 to 10 cu. ft. per sec. per sq. mi. Flood discharge was estimated to be 1340 cu. ft. per sec. per sq. mi. At Charleston, S. C., 5.0 in. of rain fell in 11 hr., Sept. 11, 1912, of which 3.0 fell in 2 hr. on Goose Creek Watershed (49.1 sq. mi.) (J. H. Gregory); on July 15, 1916, over 16.3 in. fell in 24 hr., intensity at one time was 2 in. in 10 min. and maximum flow approximately 20,000 cfs.[14]

Hawaii. Floods on small Hawaiian watersheds are recorded by Lippincott,[21] as 2810 cfs. per sq. mi. for 1.1 sq. mi. to 1640 for 1.8 sq. mi. 1.5 sq. mi. had a sustained flood at rate of 833 cfs. for 11 hr.

Sediment.† A stream through an alluvial valley tends to charge itself with sediment, which remains in exact proportion to the power of the water to carry it. Although the scouring capacity of a stream theoretically varies with the square root of the velocity, it is also largely dependent on the depth of flow (due to the increased weight).‡,[15]

Silt volume of Colorado River[31] is 100,000 acre-ft. per year, of which one-fifth comes from the Gila. Depositions in the delta region led to the alternate discharges into Salton Sea and the Gulf.

* See also *E. N.*, July 27, 1916, p. 183.
† See also p. 106.
‡ See studies by Gilbert, *Prof. Paper* 86, U. S. Geological Survey, 1914.

Table 21. Discharge and Sediment of Large Rivers[16]

River	Drainage area, sq. mi.	Mean annual discharge, sec. ft.	Sediment		
			Total annual, tons	Ratio, by weight to discharge	Depth over drainage area, in.
Potomac	11,043	20,160	5,557,000	1:3,575	0.00433
Mississippi	1,214,000	610,000	406,250,000	1:1,500	0.00288
Rio Grande*	30,000	1,700	3,830,000	1:291	0.00110
Uruguay	150,000	150,000	14,783,000	1:10,000	0.00085
Rhone	34,800	65,850	36,000,000	1:1,775	0.01071
Po	27,100	62,200	67,000,000	1:900	0.01139
Danube	320,300	315,200	108,000,000	1:2,880	0.00354
Nile	1,100,000	113,000	54,000,000	1:2,050	0.00042
Irrawaddy	125,000	475,000	291,430,000	1:1,610	0.02005

* See report by Follett to International Boundary Commission, *E. N.*, Jan. 1, 1914, p. 18.

Table 22. Quantities of Solids Carried in Suspension by Several American Rivers. In Parts per Million†

River	Place	Max.	Min.	Average		Median‡	Authority	Remarks
				Quan.	Yrs.			
Merrimac	Lawrence	1100–1500(a)	5	7.5	7	---	Clark	(a) A few days in 2 or 3 high floods in 20 yrs.; mostly fine river silt; not within 7 yrs. 1908–1914 used in average.
Potomac	Great Falls.	3000(c)	6	119 (b)	8(a)	---	Hardy	(a) Jul. 1, 1906–Jun. 30, 1914. (b) Min. average for a year, 41; max. 208. (c) Lowest max. for a yr., 1500. (From daily turbidity readings; coef. of fineness is about one.)
Allegheny	Pittsburg	3900(a)	1(a)	57	6	40	Drake	Turbidities are given. Coef. of fineness about 0.8. (a) Infrequent. Aver. susp. solids 1914 = 36.
Scioto	Columbus	2000	3	69	5(a)	20(b)	Hoover	Turbidities are given: (a) 1909–1913; (b) for 1914.
Ohio	Cincinnati	5000(d)	5(e)	192 (c)	7(a)	60(b)	Ellms	Turbidities are given: (a) 1908–1914. (b) For 1914; average for that yr. 120. (c) From daily determinations. (d) Has occurred a number of times. (e) Frequent.
Ohio	Louisville	6000(b)	15(c)	235	4(a)	---	Leisen	Turbidities are given. (a) 1910–1913. (b) Once in 1913; 5000 other years. (c) Monthly average 1911; max. monthly average, 1913, was 975.
Mississippi	St. Louis	6560	3	1400	7(a)	---	Wall	(a) May, 1905–Aug., 1912. Average turbidity, 1906–1913 was 1340; suspended solids, 1470.
Mississippi	New Orleans.	2400	55	600	6(a)	---	Earl	(a) 1909–1914. Turbidities are given.

† Annual municipal reports give similar data.
‡ By median is meant magnitude of item midway in list of all items in group considered, when these items are arranged in order of magnitude.

STREAM-FLOW CALCULATIONS

Determinations of minimum and maximum flows and their durations are generally based on stream gagings, but may be calculated roughly from rainfall data or from records of streams similarly conditioned. Long-term records are the more desirable, and calculations from rainfall or other data should be carefully scrutinized for probability, by comparison with other records, before use.

Variations in rainfall and run-off records are the rule; the task of the hydrologist is to fix reasonable limits for the design of structures. It is established that max. annual − aver. annual > aver. annual − min. annual.[7b] Hall has outlined a method of studying probable variations in annual run-off in *T. A. S. C. E.* Vol. 84, 1921, p. 191.

Length of Record Necessary for Determining Stream-flow. By study of long records for various streams in United States, Hoyt*,[17] of Geological Survey, concludes: (1) Values for 1 year vary so much from the grand mean that estimates based thereon are liable to be in great error; (2) while 5-year period in majority of cases gives values within 10 per cent. of the grand mean, there are periods when the difference may be even 20 or 30 per cent.; probability of a 5-year period comprising both a year of average low and a year of average high water is small; (3) a 10-year period with few exceptions gives values within less than 10 per cent. of the grand mean; furthermore, there occur in practically each 10-year group a year of average low and one of average high water; while this low and high may not be extreme, they give conditions to be expected except in abnormal years, which as a rule occur only once in many years. These deductions do not hold for streams in arid and semi-arid regions, where range is very great from year to year.

The average run-off of Croton River for 54 years, 1868 to 1922, was 399 mgd.; from 1869 to 1886, 18 years, flow averaged 346 mgd.; for the next 18 years, 449 mgd; and for next 18 years, 402 mgd.† This shows the futility of attempting to use even reasonably long run-off records (5 to 15 years) as a basis for computing the safe yield of a watershed, unless these records are compared with longer ones of similar watersheds, allowance being made for the differing conditions. Although the average of a 40-year period may differ from that of a 60-year period, there is a strong probability that when the record has covered 80 or 100 years, a long enough period to complete more than one cycle of wet and dry years, the average will not be far from that for 40 years. (For rainfall data, see p. 12; for run-off data, see pp. 47 and 65.) Area of Croton Watershed was taken at 360.4 sq. mi. in calculations previous to 1916, and at 375 sq. mi., subsequently.

Rainfall data have been used to extend meager run-off records. Meyer's method is outlined in Table 25; Grunsky's method in *T. A. S. C. E.*, Vol. 85, 1922, p. 88; and Justin's method in *T. A. S. C. E.*, Vol. 77, 1914, p. 346. A method of calculating in absence of rainfall records is outlined by Crawford in *Elec. World*, Dec. 4, 1920, p. 1111. Horton[29] stresses the consideration of interception of rainfall by growing crops, which his experience shows may reduce the precipitation reaching the ground 15 to 35 per cent.

* See Sargent, *E. N.*, Dec. 3, 1914, p. 1119.
† For equivalents of the several run-off units used above, see p. 816.

It is the very high percentage of run-off from the difference in precipitation which accounts to a considerable extent for the difference in the run-off per square mile of different drainage areas in Table 24.

Minimum flows can be established only by careful analysis of long-term records. Wells[18] has prepared Fig. 12 to show the fallacy of picking minimum flows from records as short as 10 years; also the possible discrepancy between annual minimum and daily minimum flows is brought out.

Minimum Run-off from Small Watersheds. Small watersheds have less run-off per sq. mi than large ones.[19] Minimum flow of seven streams in New Jersey, each with a catchment area of over 2000 sq. mi., was 0.182 cfs. per sq. mi.; twelve streams with areas between 200 and 2000 sq. mi. yielded 0.161; and twelve streams having areas less than 200 sq. mi. showed 0.094 cfs.

Croton Flow in Dry Years. Fom 1868 to 1920, inclusive, the flow of the Croton River averaged 399 mgd , or from an area of 375 sq. mi. averaged 1,064,000 gal. per day per sq mi. The following average flows occurred in successive dry years: driest calendar year on record to 1920 inclusive, 209 mgd ; 2 consecutive years, 266 mgd.; 3 years, 287 mgd.; 4 years, 291 mgd.; 5 years, 299 mgd.; 6 years, 317 mgd.

Maximum flows may be approximated by curves on p. 61. High water marks are the chief basis of estimating flood flows. Stevens[20] warns that "there is no such thing as a dependable high water mark." Personal testi-

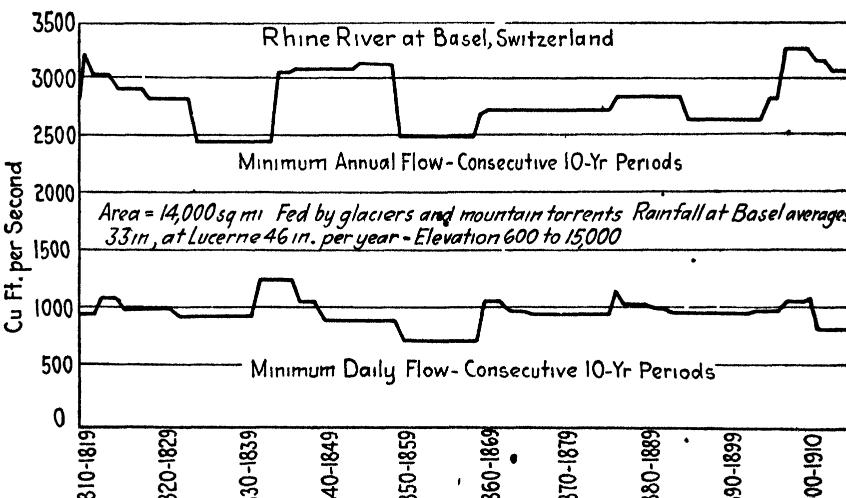


FIG. 12.—Fallacy of 10-year records.[18]

mony of old residents is unreliable. Flood run-offs from small watersheds are higher per sq. mi. than from large ones.

Formulas* **for run-off** should be used for rough approximations only

Fanning: $Q = 200\ (D_m)^{5/6}$

Ryves: $Q = C\ (D_m)^{2/3}$ (India).

Dickens: $Q = C\ (D_m)^{3/4}$

Hearn:† $Q = 640\ D_m\ (4 - \log_e D_m)$

* In 1901 Report on Barge Canal, New York State, Emil Kuichling epitomizes formulas proposed for obtaining maximum flood flows. See also p 93

† For Indian areas below 10 sq mi.

in which D_m = catchment area in sq mi.
Q = discharge in sec. ft.
C = coefficient.

In regions of maximum recorded rainfall of 3 to 6 in. in 24 hr., coefficient C is:

	Dickens	Ryves
Flat country	200	400 to 500
Mixed country	250	...
Hilly country	300	...
Maximum rainfall	300–350	650

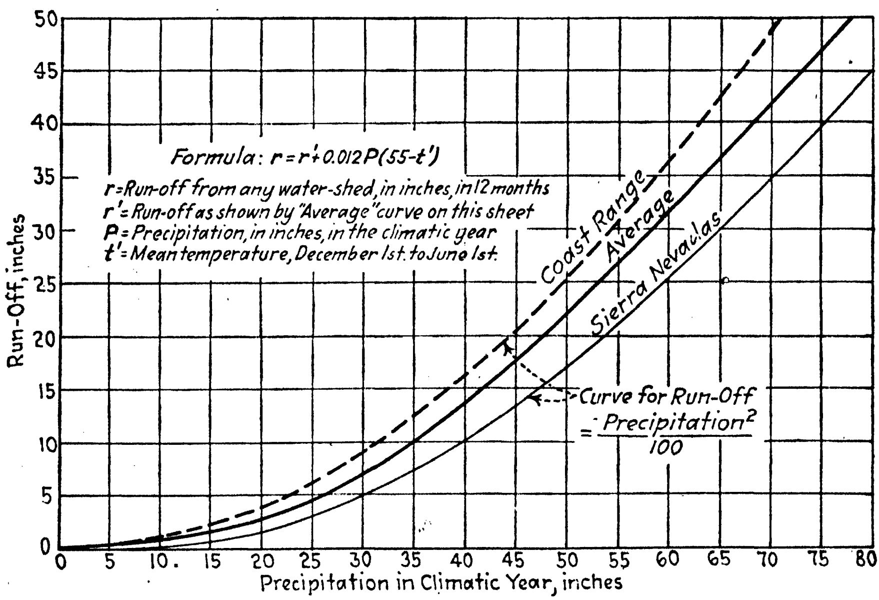


Fig. 13.—Probable rainfall-run-off curves for California.[25]
(Curves for Coast Range and Sierra Nevadas are by Grunsky in T. A. S. C. E., Vol. 51, 1908, p. 512.)

Estimates of available run-off should be based on the minimum for a long series of years, as the water value of a given drainage area during dry years is obviously only the annual run-off of these years. The annual run-off or yield on a given stream occasionally varies nearly 100 per cent. for the same annual rainfall. In New York and New England, the average run-off is about 45 per cent. of the rainfall; 52 years' observations on the Croton watershed show that the average run-off is about 47 per cent., but the years of lowest rainfall show as low as 31 per cent. The dry condition of the ground in times of drought keeps much water in the interstices. From 1868 to 1920, the Croton year of lowest rainfall showed a yield of 603,000 gal. per day per sq. mi.; next driest year, 760,000; average for 52 years, 1,064,000. The safe average permissible draft between these limits should be chosen, as say 850,000. The year of maximum yield, 1901, averaged 1,693,000. Calendar years are referred to in this paragraph.

Table 23. Run-off of Croton Watershed, June to Dec. each Year, 1868–1920, Compared with Total for Same Years; also with Rainfall

Year	Run-off, inches, June to Dec.	Per cent. of total run-off for year	Per cent. of rainfall for same months	Year	Run-off, inches, June to Dec.	Per cent. of total run-off for year	Per cent. of rainfall for same months	Year	Run-off, inches, June to Dec.	Per cent. of total run-off for year	Per cent. of rainfall for same months
1868	18.2	55.1	59.1	1886	4.0	20.6	15.8	1904	9.7	38.7	31.0
1869	9.3	40.2	32.4	1887	11.5	44.0	30.2	1905	6.6	33.1	22.3
1870	3.1	16.3	13.8	1888	15.7	45.1	43.8	1906	6.4	31.5	23.5
1871	10.3	52.9	33.6	1889	19.4	62.7	49.6	1907	16.8	57.6	35.5
1872	8.6	50.9	30.0	1890	11.1	43.5	34.9	1908	2.9	14.6	15.2
1873	7.7	31.4	30.5	1891	3.9	16.2	15.8	1909	3.7	18.5	14.3
1874	5.2	20.8	22.7	1892	6.3	36.1	23.5	1910	2.9	15.3	14.1
1875	10.8	43.6	38.2	1893	9.2	32.1	32.3	1911	10.6	53.7	33.8
1876	*2.8	13.5	14.2	1894	8.8	43.6	30.2	1912	5.2	22.7	21.3
1877	8.2	41.6	24.8	1895	3.5	22.4	13.9	1913	8.6	30.7	30.8
1878	14.7	55.0	41.6	1896	6.0	26.3	23.7	1914	3.5	14.0	16.4
1879	6.0	31.0	20.3	1897	12.3	48.2	35.5	1915	10.2	39.3	31.5
1880	1.9	15.6	8.8	1898	12.3	41.6	36.6	1916	6.9	25.1	27.1
1881	4.6	24.5	18.7	1899	5.1	22.5	20.4	1917	5.8	30.7	27.6
1882	9.4	38.4	30.8	1900	5.4	22.9	22.6	1918	3.8	21.2	15.7
1883	3.0	23.3	11.5	1901	17.3	47.2	44.6	1919	10.9	26.4	36.7
1884	7.8	33.3	26.1	1902	11.6	34.6	36.2	1920	14.8	22.1	21.8
1885	6.0	34.7	21.4	1903	17.3	49.6	43.3				

Lowest month, Sept., 1881: −0.03 in.; evaporation exceeded rainfall. * Lowest 7 months.

Table 24. Comparison of Run-off in Years of High and Low Precipitation[24] (Calendar Years)

Drainage area	Period	Average precipitation, inches	Average run-off, inches	Per cent. of run-off
Wachusett (Mass.) (1897–1914).	Highest 4 yrs.....	54.5	29.3	53.7
	Lowest 4 yrs.....	38.7	17.1	44.2
	Difference.....	15.8	12.2	77.0
Sudbury (Mass.) (1875–1914).	Highest 6 yrs.....	55.2	29.1	52.7
	Lowest 6 yrs.....	36.4	13.5	37.2
	Difference.....	18.8	15.6	82.9
Croton (N. Y.) (1868–1913)...	Highest 6 yrs.....	59.4	31.9	53.7
	Lowest 6 yrs.....	40.6	18.5	45.6
	Difference.....	18.8	13.4	71.3
Average Perkiomen, Neshaminy, and Tohickon (Penna.).	Highest 4 yrs.....	55.0	29.2	53.0
	Lowest 4 yrs.....	42.3	19.5	46.0
	Difference.....	12.7	9.7	76.2
All sources................	Highest 20 yrs....	56.3	30.0	53.3
	Lowest 20 yrs....	39.3	16.9	43.1
	Difference.....	17.0	13.1	77.0*

*Obtained by dividing 13.1 by 17.0. and so for other differences

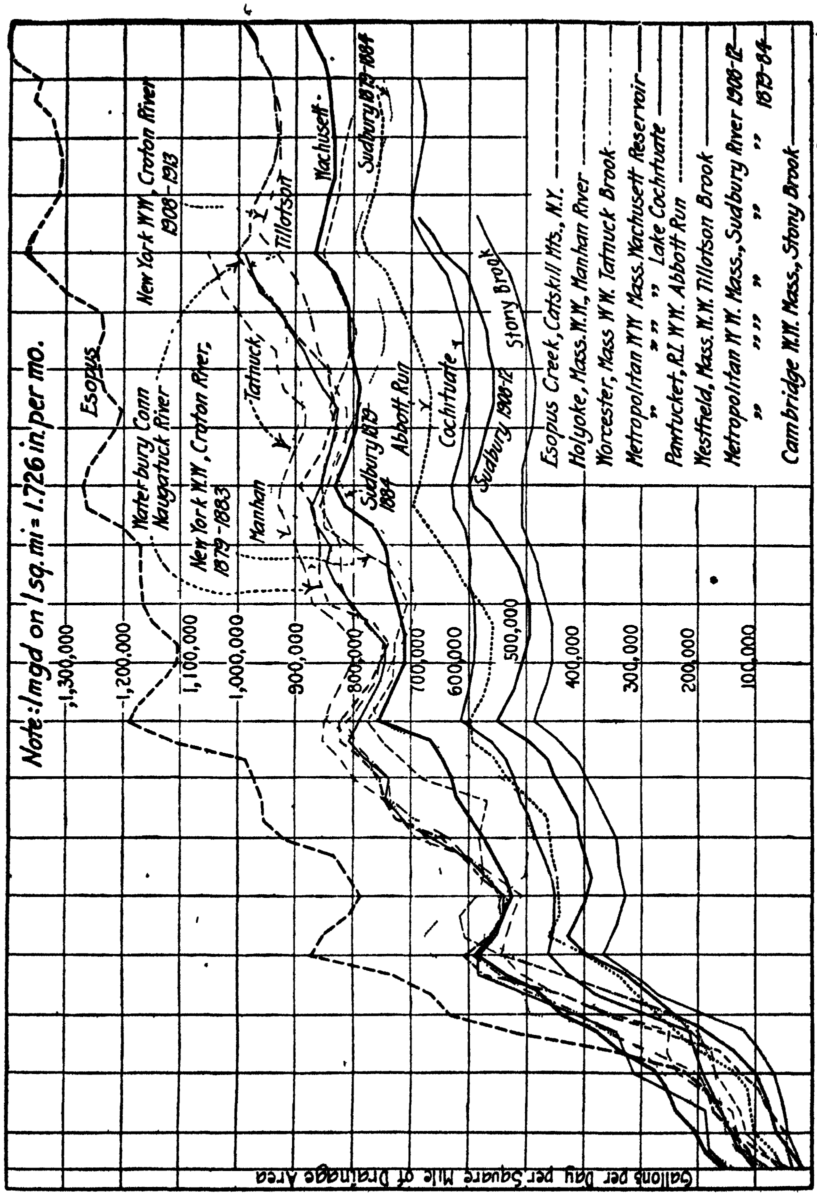


Fig. 14.—Average daily yield of one square mile of drainage area, land surface only, for the driest periods of consecutive months.

The severe drought, 1908–1913, is included Curves show mean yields of drainage areas for numbers of consecutive months of minimum yield on bottom of diagram. N. E W. W. Assn, Comm on Yield of Drainage Areas, 1914 [24] See also Report of Committee on Rainfall and Run-off Measurements, J N. E. W. W. A., Vol. 35, 1921, p 151.

Table 25. Observed and Computed Physical Data for Fifteen Watersheds.

Adolph F. Meyer[27c]

Reference No.	Name of watershed (river)	Years of record	Evaporation coefficient	Area, in square miles	Observed and computed physical data—mean annual, inches: Rainfall	Temperature	Evaporation	Transpiration	Total loss	Precipitation minus total loss	Computed run-off	Observed run-off	Time of record
1	Mississippi.....	17	1.20	19,500	27.3	41	14.4	7.7	22.1	5.2	5.23	5.31	1897–1913
2	Little Fork.....	5	1.10	1,720	23.9	37	11.2	6.9	18.1	5.8	5.80 *5.15	..... *5.13	------
3	Minnesota......	5	1.25	6,300	22.7	43	14.1	7.5	21.6	1.1 *0.77	1.1 *0.77	..... *0.70	------
4	Root...........	6	1.225	1,560	31.2	45	16.5	8.6	25.1	6.1	6.10 *5.40	..... *5.20	------
5	Ottertail.......	6	1.10	1,310	23.0	40	13.5	6.6	20.1	2.9	2.80 †2.66	..... †2.59	------
6	St. Croix.......	11	1.05	5,930	30.0	41	13.1	7.0	20.1	9.9	9.90	9.60	1901–1912
7	Ohio...........	14	0.875	23,820	41.1	51	14.8	5.8	20.6	20.5	20.50	22.00	1891–1905
8	Tohickon Creek.	24	0.90	102	48.9	51	16.7	7.0	23.7	25.2	25.2	26.10	1887–1911
9	James..........	7	0.925	6,230	42.1	54	16.3	7.0	23.3	18.8	18.90	18.00	1898–1905
10	Roanoke.......	9	0.90	390	42.6	57	16.9	7.0	23.9	18.7	18.60	17.70	------
11	Tombigbee.....	9	1.05	4,400	49.2	62	22.8	8.4	31.2	18.0	18.00	17.10	1900–1909
12	Colorado.......	10	1.20	37,000	26.9	66	17.7	8.1	25.8	1.1	1.06	0.74	1900–1910
13	Sacramento.....	9	0.85	10,400	32.2	52	8.5	2.4	10.9	21.3	21.3	20.40	1902–1911
14	Pit.............	6	1.10	2,950	14.8	48	6.9	3.0	9.9	4.9 *3.87	4.9 *3.87	..... *3.92	1903–1908
15	McCloud.......	6	0.60	608	61.9	55	8.2	2.4	10.6	51.3	51.3	54.00	1902–1908

* 4 years' records. Calendar years. † 5 years' records.

1. At Minneapolis: elevations along river, 800 to 1475 ft., slope 1.5 ft. per mi.; some lake and swamp areas, but no allowance for loss therefrom except slight increase of evaporation coefficient; ⅓ under cultivation, remainder forests and brush.

2. At Little Fork, northern Minn.: elevation 1100 to 1400 ft., slope along river 2 ft. per mi.; flat topography, only few hills more than 50 to 75 ft. above plain; extensive swamps; clayey soil; generally densely wooded.

3. At Montevideo, western Minn.: slope of upper river 20 ft. per mi., lower river 0.6 ft. per mi.; flat open prairie, almost all under cultivation; large marshes with clayey soil near headwaters; water surfaces estimated 5 per cent. of watershed; transpiration limited by available moisture.

4. At Houston, southeastern Minn.: flat, to gently undulating, streams in deep V-shaped valleys; mostly cultivated, woods along streams; soil is clayey loam, causing high evaporation losses; slope of river, 6 ft. per mi.

5. At Fergus Falls, western Minn.: dotted with lakes, water surfaces totaling 15 per cent. of watershed; lightly timbered, with much land cultivated; well-sustained stream flow; prominently rolling, morainic and knolly; heavily covered with drift.

6. At St. Croix Falls, Wis.: undulating, thickly covered with glacial drift; no marshes, numerous lakes, good ground storage; elevations along river 750 to 1000 ft.; slope of river 2.5 ft. per mi.

7. At Wheeling, W. Va.: northeastern portion rolling and hilly, remainder mountainous; shallow soil; slope along Allegheny River 6.5 ft. per mi.; heavy snow and thick ice common in Allegheny Basin, but winters are less severe in upper Monongahela Valley.

8. At Point Pleasant, southeastern Pa.: slopes steep; ¾ of area cultivated; ¼ wooded and untillable; slope of creek 20 ft. per mi.

9. At Cartersville, Va.: upper portion of watershed mountainous and wooded, elevations up to 4000 ft.; remainder is rolling plateau. (See also p. 53.)

10. At Roanoke, Va.: rugged with steep slopes along river; elevations up to 3000 ft.; soil scant; little land cultivated. (See also p. 55.)

11. At Columbus, Miss.: flat country; streams have gentle uniform slope; ¼ cultivated, remainder heavily forested; deep soil, heavy loam; humidity not much higher than in northwest; average annual snowfall 4 in.

12. At Austin, Tex.: comparatively flat, but slopes pronounced; timber along streams and at higher elevations; soil deep and not sandy; humidity and wind differ but slightly from those in northwest.

13 to 15. At Red Bluff, Cal.: about ½ covered with timber, remainder mostly grass land; scant soil cover over lava; most of precipitation, especially at headwaters, is snow; elevations mostly 3000 to 5000 ft.; main portion of discharge comes from Pit River basin, which is relatively flat and largely pasture land.

Bibliography, Chapter 3, Stream Flow

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CHAPTER 4

GROUND WATER*

Utilization. Ground-water supplies are the chief resources of most small municipalities and some large cities; among them Brooklyn, Lowell, and Memphis. Development works may consist of wells (Chap. 10), springs (Chap. 11), infiltration galleries (Chap. 12), or a combination. Ground waters have important influence on the low-water flow of streams.

Occurrence. Ground water may issue from an aquifer (water-bearing formation) as a spring, or may be secured by development works. Any depression cutting the water table tends to fill with water. If open at the end, a spring-fed stream is formed (Fig. 15); if closed, a lake; and if closed and of small dimensions, a spring. Flow of water toward the depression follows hydraulic laws (see p. 77). As the hydraulic slope diminishes, flow lessens. Pondage of water flattens the slope and causes diminution of flow. Excessive pumping steepens the gradient, but reduces the depth of the water-bearing channel, and may result in lessened flow (see p. 243). At the Hempstead reservoir, Long Island, N. Y., it was found that the yield was 5.6 mgd. when the water was impounded to 14.35 ft. depth, and 8 mgd. when at only 4 ft. depth.[1a] The underflow of streams is usually very small and its velocity very low. Most streams deposit enough silt to make their banks and bottoms practically impervious.

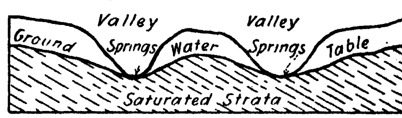

FIG. 15.

Ground-water Supplies vs. Surface Waters. Ground waters have the merits of natural filtration and of coolness in summer. Extensive use of bottled water in office buildings indicates preference for spring waters to treated surface supplies.† Waters in sub-surface storage are less susceptible to evaporation. For small supplies, ground waters can be more cheaply impounded than surface waters. Ground-water storage in the strata adjacent to a surface reservoir is often an important asset (see p. 86). Sources of ground-water pollution, on the other hand, are often difficult to detect. Ground-water supplies are dependent on the maintenance of the water-table; its recession has caused abandonment of many supplies. The durability of ground-water supplies is often open to question. C. H. Lee has indicated a method of investigation in *T. A. S. C. E.*, Vol. 78, 1915, p. 148. Deep-well waters often prove excessively hard; many are abandoned for this reason. Pumping from a well is more expensive than from the surface. To investigate ground waters and estimate the yield is more costly than for surface waters; the area is often ill-defined. Ground waters are no less inexhaustible than surface waters.

* For terminology of ground-water hydrology, see Meinzer, *Water Supply Paper*, 494, 1923.
† See "Underground Waters for Commercial Purposes," by F. H. Rector, 1918 (John Wiley & Sons, Inc.).

Replenishment of ground-water catchment areas may be by: (1) rainfall on the exposed area which penetrates to the water-bearing strata; (2) flood waters of rivers which periodically cover outcrops of the ground-water strata; (3) surface flow of water which has been stored in reservoirs for feeding the water-bearing strata by percolation; (4) percolation of irrigation water from a distant source.[2]

Investigating Ground-water Supplies. It is popularly supposed that ground waters are inexhaustible and have mysterious origins and paths of travel. The locating of possible supplies depends upon knowledge of local conditions, geology, and previous experience. Many studies by the U. S. Geological Survey are available. The futility of the divining rod is discussed in *Water Supply and Irrigation Paper* 416, 1917. Fluorescein,[3] and recently uranine have been used to trace the path of underground waters. See also p. 74. Observing well drillers will take into account surface drainage of the region, presence or absence of forest which may influence run-off, existence of springs as evidence of surplus water, and depth and character of soil overlying the rock, especially structure of the soil cover. If water-laid, and therefore stratified and regular, accurate and quick deduction can be made as to depth and availability of ground water. If material be wind-laid, and therefore less regular, deductions are uncertain; likewise in regions formerly covered by ice (glacier), where it may be still more irregular. All ground water has more or less intimate relation to surface drainage and topography.[4]

Absorption of Rainfall by Ground. Ground receives the greater part of the rainfall, probably nearly 80 per cent., in Eastern United States, and 90 or 95 per cent. in much of the West. Water that enters sands and gravels generally moves toward streams, but, in regions where the rainfall is small, gravels may absorb water from streams which rise in regions of greater rainfall. In the Eastern United States, about 5 to 6 in. of the rainfall goes through this seepage process; in England, 10 in. is estimated (see also p. 260). This diffused seepage maintains stream-flow long after the direct flow of the rain-water over the surface has ceased.[5]

Most favorable ground-water areas are the deep water-bearing strata beneath broad plains that were created during the glacial epoch by out-washes of coarse sands and gravels from the terminal moraines of glaciers.

Depth of Water-table. The upper limit of water-bearing strata is known as the water-table. Its depth varies with: (*a*) porosity of the catchment area; (*b*) amount of rainfall; (*c*) topography; (*d*) pumping rate when developed. In general, the contour of the land bears a more or less definite relation to the contour of the water-table. The underground flow is normal to these contours. In regions of low rainfall and low relief, the water-table is deep-seated and relatively horizontal. In regions of greater rainfall and greater relief, it is relatively near the surface and tends to follow the topography, but slopes to a less degree; it is nearer the surface in lowlands.

Relation to Vegetation. Investigations have shown that when the ground-water surface is 7 to 10 ft. or more below the surface of coarse soils, no moisture reaches the surface or the roots of vegetation through capillary action. King found the water-table underneath crops lower than under bare areas adjoin-

ing,[6] and Ototzky[7] found a marked depression underneath forests as compared with the water-table in near-by cleared areas of similar geologic formation.*

Annual Fluctuations of Ground-water Level,[9a] high in summer, low in winter, are caused by the quantity of rainfall reaching the plane of saturation. Frozen ground minimizes the absorption of surface waters, although King's experiments in Wisconsin showed conclusively that some water percolates into frozen ground, possibly through shrinkage cracks. Causes[10a] of fluctuation of ground water-table on Long Island are (1) natural—rainfall, sympathetic tides, thermometric changes, barometric changes;† (2) artificial—dams, pumping

Statistics by Magee[11] for 28,906 wells lead him to conclude that the development of the United States has resulted, outside the irrigation region, in lowering the water-table no less than 9 ft. Horton doubts the soundness of

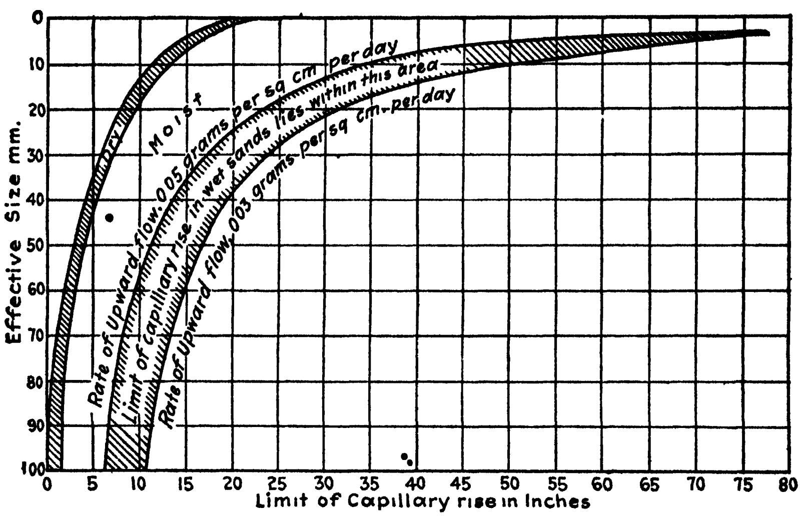

FIG. 16.—Relation of capillary rise to effective size.[9b]

this deduction.[12a] Tests on Long Island indicate that the deeper below the surface and the higher above sea level the water table is, the less rapidly it responds to rainfall influence. Retardation is out of proportion to the thickness of the unsaturated beds above the water-table.

Underground Water in Rocks.‡ Experience does not prove the assumption that all rocks are saturated below a moderate depth. In the Pennsylvania and New York oil regions, rocks practically destitute of water are encountered at a few hundred feet. These include coarse-grained sandstones capable of holding large quantities, but which are dry. Very rarely is fresh water found below the dry rocks. Wells have been drilled several thousand feet deep without encountering water below the first few hundred feet. These facts show the fallacy of the idea that there is plenty of water if one only goes deep enough. That great underground lakes exist is extremely improbable. Deep-

* *Bull* 852, 1920, U S Dept of Agri also discusses this relation

† Anthony cites the direct relation between the well levels of deep-seated mineral waters at Saratoga, and barometric levels.[8]

‡ See also p. 75.

ening of wells rarely adds to the hydraulic head, but commonly increases the salt content to a degree causing abandonment.*

Yield is a function of the rate of replenishment, and, to a less degree, of the extent of underground storage. The normal yield of the ground-water catchment area of the Ridgewood (Brooklyn, N. Y.) system, is about 900,000 gal. per day per sq. mi., or 43 per cent. of the average rainfall. European watersheds of similar character, on which the rainfall is much less than on Long Island, have averaged 40 to 50 per cent. of the rainfall for years.† Most

Table 26. Relation between Rainfall and Ground-water Yield

Place	Annual rainfall, in. (1)	Average yield, in. (2)	Ratio (1):(2)
Long Island, N. Y[13]	45	14–24	0.30–0.50
Holland, Mich[13]	34	8.7	0.26
Philadelphia[13]	43	5–8	0.12–0.19
The Hague, Holland[31]	27	15.3	0.57
Kohlfrist tests[2]	....	....	0.285
Brussels, Belgium[31]	28	6–10	0.22–0.35
Munich, Bavaria[31]	47	30	0.65*

*Approximate.

wells in the immediate vicinity of large water bodies obtain much of their supply by infiltration therefrom. Flooding, termed "flood spreading,"‡ of gravelly land adjacent to wells has been practiced in the West,[14] and also at Greenfield, Mass., Breslau, Germany, Brooklyn, N. Y., Brookline, Mass., Charnitz, Germany, Stockholm, Sweden, Frankfort, Germany,[15] and elsewhere to increase the yield.[3] During a single flood at Breslau, rise of the water-table indicated that 1325 mg. had seeped into the ground.[13]

Yield tests are to be preferred to the theoretical application of formulas to uncertain conditions. Tests commonly consist of sinking one well and noting the rate of depression of the water-table under pumping conditions for a week. At South Bend, Ind.,[16] a central, 12-in. well, surrounded by 2½-in. wells on a 6-ft. radius, 155 ft. deep, was dosed with 200 lb. of salt and basic fuchsin. The 2½-in. wells were fitted with electro-couples connected to the lighting circuit through an ammeter. Readings and samplings hourly gave a double record of the rate and direction of flow.§ Chemicals were used in tests at Des Moines.‖,[17]

Velocity Measured.[18a] Slichter¶ observed an average velocity of 7.4 ft. per 24 hr., and a maximum velocity of 22.9, of underflow in Arkansas River sand.[18a] G. E. P. Smith found a velocity of 400 ft. per day in the underflow of an Arizona stream.[18a] Character of deposit has large influence. On Long Island, Slichter found variable rates of 2 to 96 ft. per day.[10b]

* See also p. 245.
† See also p. 260.
‡ See "Underground Water Storage in the Santa Ana Cone," *E. N. R.*, Oct. 27, 1921, p. 683; and *T. A. S. C. E.*, Vol. 82, 1918, p. 802; also, "The Future Water Supply of San Francisco; a *Report* to the Secretary of the Interior and the Advisory Board of Engineers of U. S. Army, by Spring Valley Water Co., Oct. 31, 1912.
§ See also, "Yield of Underground Sources Determined," by W. S. Coulter, *Fire and Water Eng.*, September, 1923.
‖ See also p. 78.
¶ For SLICHTER's electrical method, see *Water Supply and Irrigation Paper*, 67, 1902, p. 50.

Artesian Conditions.* In order that a well may flow, the following conditions must be satisfied: (1) sufficient rainfall; (2) relatively porous beds suitably exposed to collect and transmit the water; (3) less porous or relatively impervious layers so placed that they may confine the water collected; (4) the level of the ground water at the source should be at a sufficient height above the mouth of the well to compensate for loss of head due to resistance and leakage.

An artesian well in Chicago, drilled in 1864, had its original head 80 ft. above ground; water level is now 220 ft. below ground, a recession of 300 ft.[19] There is great loss of artesian water from open wells, and from casings of old oil wells,[20] where it is allowed to run to waste. Depletion of artesian basins has occurred in Australia and southern California, and wells at higher elevations have gone dry.[12b]

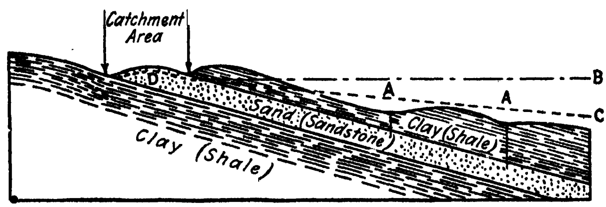

FIG. 17.—Ordinary conditions producing artesian wells. A, wells; B, head of water if there is no loss by resistance and leakage; C, actual head or hydrauilc gradient; D, ground-water table at outcrop.

WATER-BEARING FORMATIONS[4]

Beds of sand and gravel[21] are very porous, as much as 30 per cent. of the volume of some being free space, so that saturated layers penetrated by wells yield copious supplies. This water is, in most places, of good quality, but in some wells is greatly mineralized from the more soluble rock fragments that constitute the deposits. In passing downward through finer sands, surface water may be filtered naturally. In coarse sands and in gravels, water passes downward more rapidly and conditions are less favorable for filtration. In general, water from sands and gravels far below the surface is pure. Because sands and gravels yield readily, wells close together may affect one another, those which derive supply from the lowest points drawing from the shallower wells. To procure permanent supplies, a well should penetrate below the level to which the water surface sinks in driest seasons. Wells in sand and gravel should have large enough diam. to avoid clogging around casing, which seems inevitable in wells of small diam.

Pure clay is nearly impervious to water and contains little or none that can be utilized; water is frequently reported in clay, but as a rule it comes from more or less sandy layers within a clay bed. In some places, sand that approaches clay in fineness and is mistaken for clay yields much water. Clay is of great importance as a confining layer to porous sands. When necessary to obtain water from clay, the well should be as large as practicable and deep enough to provide ample storage, for clay yields a small quantity, and very

* For exhaustive discussion, see "The Determination of Safe Yield of Underground Reservoirs of the Closed-Basin Type," by C. H. LEE, *T. A. S. C. E.*, Vol. 78, 1915, pp. 148–251.

slowly. Dug wells are usually most satisfactory where clay is near the surface, but should be carefully covered and protected from pollution.

Till is a heterogeneous mixture of clay, sand, gravel, and bowlders. In texture it ranges from very pervious to impervious, according to content of sand or of clay. In few places is it definitely bedded. Water generally occurs in it in minute, more or less tubular, channels, but occasionally is distributed through interstratified sandy beds. In finer, more loamy phases, the supply is not abundant, but coarser portions furnish water more plentifully. In the aggregate, till yields a large quantity, and, where sufficiently thick, forms a most convenient and accessible source, but because so irregularly disposed the success of wells varies greatly. Water is usually close to the surface, but better can be obtained by casing off upper water-bearing beds and extending the wells to greater depth. In general, wells of large diam. are most satisfactory. Till is widely distributed over Northeastern and North-central United States in areas covered by glacial drift. The southern border of glacial deposits extends from Martha's Vineyard, Mass., through Long Island and northern New Jersey, across northern Pennsylvania into southwestern New York; thence southwestward across Pennsylvania and Ohio, southern Indiana, Illinois, and central Missouri, and northwestward into Canada.

Sandstone is, of solid rocks, the best water bearer. Water from sandstone is of better average quality than that from any other material except sand and gravel. In the Appalachian Plateau*—the bluegrass district of Kentucky, Nashville basin, and Cumberland and other plateaus of Tennessee, West Virginia, western Pennsylvania, and New York—sandstone is the chief water-bearing sedimentary rock. Porous sandstones also underlie the great plains of the Dakotas, Kansas, and Nebraska, and yield artesian flows throughout extensive areas. Over much of the region farther south they also furnish abundant underground water. In some places *conglomerate* yields considerable water, although absorptive capacity is not so great as of sandstone; it is not so widely distributed. *Quartzite* is a metamorphosed sandstone, the spaces between grains being filled with hard, siliceous matter. Because of this filling of the pores, there is little chance for water, hence it is not commonly an important source. Old sandstones, shales, and other sedimentary rocks, altered to quartzites or slates, lie near the surface in the Appalachian Mountains, Ozark and Superior highlands, and Rocky Mountains, and on the borders of these higher lands yield some water, but generally both occurrence and quantity are uncertain.

Shale is a poor water bearer, but may yield from bedding, joint, and cleavage planes, and other crevices; most important as a confining layer to prevent escape of water from interbedded porous sandstone.

In **limestone** water occurs mainly in channels and caverns formed by solvent action along joint or bedding planes. These are very irregularly distributed and location can seldom be determined by examining the surface, but most deep wells encounter one or more such passages at relatively slight depth. Adjacent wells, only a few feet apart, may obtain very different results. Water is hard, and likely to be polluted, since much of the water in limestone

* For "Absence of Water in Certain Sandstones of the Appalachian Oil Fields," see *Economic Geology*, Vol. 12, 1917, p. 354.

has found its way downward through sink holes, and carries surface wash. Water-bearing limestones occur in Southern and Southeastern United States and in the Appalachian Mountains, but owing to poor quality, recourse is usually had to springs or to wells sunk in other rocks.

Granite and gneiss are dense and have very small pore spaces. *Schist* may contain water that has penetrated along foliation planes. Most wells in crystalline rocks obtain water, if at all, within 200 or 300 ft. of the surface, and it is generally useless to go deeper than 500 ft., although in some places, as at Atlanta, Ga., water supplies have been obtained at depths as great as 1600 ft. Joints in crystalline rocks usually form complex systems of intersecting planes, and polluted water may pass from the surface along joints until finally it reaches the well at a depth of many hundred feet.

Basin and Stream Deposits.* Along northern portion of Atlantic coastal plain are extensive lake and stream deposits of clay, sand, and gravel, in which water is usually plentiful and good. In lower Mississippi Valley fine silts of the flood plain are saturated a short distance below the surface and furnish abundant supplies to small wells; the chief drawback is the unusually small mesh of screens necessary to exclude fine sand. In most places a lens of coarser sand or gravel can be located, and drawn on. In many parts of the arid region of Western United States great basins or valleys are deeply filled with sediments brought down by streams. In some places these contain water at shallow depths, and form the chief source of supply throughout much of the Great Basin region of Utah, Nevada, southeastern Oregon, and southeastern California. In the more favorably situated of these desert valleys, notably Coachella Valley, in southeastern California, the deeper water of the unconsolidated deposits may be under sufficient artesian head to yield flowing wells. In the great central valley of California and on the coastal slopes of the Pacific States are deep alluvial deposits of importance as sources of water. Those of the coastal plain of southern California have been extensively tapped to obtain water for irrigation, and water is drawn from similar deposits around Puget Sound.

FLOW OF UNDERGROUND WATERS

Laboratory Experiments Inadequate. Laboratory experiments on flow through unassorted sands do not consider the modifying influences of frost, vegetation, or the intermittent and variable intensity of application of water. Permeability of soil has been found to be one-fourth as much as fine sand; when the organic matter was burned out, it was about one-half as permeable. A disturbance of the medium has a marked effect on the rate of flow. Even a very slight scratching of the surface seriously affects the discharge. With pure blow sand, percolation decreased from 15 to 5 cu. ft. per sec. due to a slight disturbance of the surface. The manner of stratification also has a modifying influence.

Formulas.† The principal formulas now in use are:

Hazen's:[25] $$V_m = C_p d^2 S \frac{t_f + 10}{60}$$

* Preglacial gorges have been developed, as at Virden, Ill.[22]

† Hazen[23] and others have repeatedly warned against injudicious use of formulas. For New England soils, Johnson[24] considers such calculations worthless.

Slichter's:[1]	$Q = 0.2012S \dfrac{d^2A}{uK}$.
Darcy's:	$Q = C_pSA$.
Forchheimer's:	$S = aV_m + bV_m^2$.
Lueger's:[26]	$V_v = 283\ dS$.

a = a constant, the conditions governing it not being explained.
b = a constant, the conditions governing it not being explained.
C_p = constant, depending on porosity of the medium.
d = effective diameter of sand grain, millimeters; varies from 0.1 for very fine sand to 3.0, fine gravel. See also p. 703.
A = area of total cross-section yielding water, in sq. ft.
A_v = area of the void section, sq. ft.
S = slope head or difference of elevations of the water at the two points considered ÷ distance traveled, measured along path of travel = $\dfrac{h}{L}$.
K = constant, dependent on porosity of the medium.
Q = cu. ft. per min. of water yielded by the entire cross-section.
t_f = temperature of the water, in °F.
T_c = temperature of the water in °C.
u = a constant known as the coefficient of viscosity, which takes account of the friction between particles of liquid.
V_m = velocity in meters per 24 hr., at which water issues from the entire cross-section.
V_d = V_m expressed in ft.
V_v = velocity, in ft. per 24 hr., at which water travels through the voids.

Slichter's formula may be written:

$$V_d = 289.728Sd^2 \div uK;$$

and Hazen's:

$$V_d = \frac{292}{89} C_pd^2S \frac{t_f + 10}{60}.$$

Table 27. Porosities of Some Media Encountered by Ground Waters[28]

Rock or earth	Quarts of water per cu. ft.	Porosity, per cent. Minimum	Maximum	Average
Granite, schist and gneiss	0.003–0.06	0.02–0.4	0.6–1.9	0.2–1.2
Gabbro	0.06†	———	———	0.8
Diabase	0.07†	0.9	1.1	1.0
Obsidian	0.04†	———	———	0.5
Sandstone	0.5–1.5	3.5–4.8	22.8–28.3	10.2–15.9
Quartzite	0.01–0.06†	———	———	0.2–0.8
Slate and shale	0.30†	0.5	7.6	4.0
Limestone, marble, dolomite	0.06–2.5	0.5	13.4	4.8
Chalk	2.0	———	———	53.0
Oolite	0.54†	3.3	12.4	7.2
Gypsum	0.19†	1.3	4.0	2.6
Sand (uniform)	2.5	26.0	47.0	35.0
Sand (mixture)	2.85†	35.0	40.0	38.0
Clay	3.35†	44.0	47.0	45.0
Soils	4.12†	45.0	65.0	55.0

† Computed from average porosity.

Application of Lueger's Formula. $V_v = 283\ dS$. An underground water current 3000 ft. wide and 3 ft. deep, has a slope of 0.001, passing through a medium of 25 per cent. porosity, with $d = 2.0$.

$$Q = A_v V_v = 0.25 \times 3000 \times 3V_v = 0.25 \times 3000 \times 3 \times 283 \times 2 \times 0.001$$
$$= 1275 \text{ cu. ft. per day.}$$ [26]

Application of Hazen's Formula.* In considering flow of ground waters, refinements like influence of temperature may be eliminated. Thus Hazen's formula becomes:

$$V_d = 8200\ d^2S.$$

Velocity of ground water, through fine sand having effective size of 0.1 mm. and general slope of ground-water surface of 5 ft. per mi. (hydraulic gradient = 0.001), would be 0.082 ft. per day. Velocity through gravel having effective size of 3 mm. and general slope of water surface of 53 ft. per mi. (hydraulic gradient = 0.01) would be 738 ft. per day, or approximately 0.0085 ft. per sec. Report of Mass. State Board of Health, 1892, contains results of Hazen's experiments to determine velocities of water through screened gravel, assuming 40 per cent. porosity.

The formula $V_d = 8200d^2S$ gives much larger results than Table 28. To modify the formula to produce the results given in the table would involve a variation of coefficient between 6222 (for d = 3 mm. and s = 0.0005) and 940 (for d = 35 mm. and s = 0.01).

Table 28. Velocity of Water in Feet per Day, in Screened Gravel, Assuming 40 Per Cent. Porosity[18b]

Slope S	Effective size, millimeters									
	3	5	8	10	15	20	25	30	35	40
0.0005	28	82	164	246	410	656	902	1,230	1,640	2,050
0.001	57	172	335	475	820	1,210	1,680	2,250	3,030	3,690
0.002	115	328	639	902	1,550	2,250	3,030	3,930	4,830	5,820
0.004	221	631	1,230	1,700	2,870	3,930	5,000	6,060	7,130	8,200
0.006	336	918	1,690	2,250	3,690	5,080	6,390	7,620	8,930	10,100
0.008	443	1,160	2,060	2,780	4,340	5,900	7,380	8,930	10,400	11,800
0.01	549	1,410	2,460	3,150	5,000	6,800	8,440	10,000	11,500	....

Hazen's C_p allows for porosity; the formula[25] applies to sands with effective diam. from 0.1 to 3 mm.; for gravels of larger size, friction varies in ways not expressible by a general formula.[18c] As the size increases beyond this point, the velocity with given head does not increase as rapidly as the square of effective size; and with coarse gravels, the velocity varies as the square root of the head instead of directly with the head, as in sands. The influence of temperature also becomes less marked with coarse gravels. Whenever properly applied, the Hazen formula has proved reliable; C_p rarely falls below 400, even for dirty sands, and rarely rises above 1200, lying between 700 and 1000 in most cases. This formula is not to be applied to clay, hardpan, soil, etc.* Voids may be measured by ascertaining the volume of water contained in a sample. This method is subject to errors due to air entrained in the water, and the absorbent action of the stone. Better, weigh a known volume and from the known weight of an equal volume of solid stone, compute voids.

* See footnote p. 77.

Uniformity Coefficient. The Massachusetts tests indicated that the finer 10 per cent. of the sand grains control the transmitting power. The uniformity coefficient is found by dividing the size of grain separating the coarser 40 per cent. from the finer 60 per cent. of the particles, by the size of grain separating the finer 10 per cent. from the coarser 90 per cent. A rough estimate of the open space can be made from the uniformity coefficient. Sharp-grained particles having a uniformity coefficient below 2 have nearly 45 per cent. voids as ordinarily packed, and sands having a coefficient below 3, as they occur in banks, or artificially settled in water, will usually have 40 per cent. open space. With more mixed materials, the closeness of packing increases until, with a uniformity coefficient of 6 to 8, only 30 per cent. open space is left. With round-grained water-worn sands, the open space has been observed to be from 2 to 5 per cent. less than for corresponding sharp-grained sands.[25] In tests on materials for Cold Spring dam[30] it was found that a small effective size and a large uniformity coefficient were a fairly reliable indication of small flow. The proportion of voids was found to be of little use in ascertaining the permeability of a medium. Probably one of the most influential factors in determining porosity of soils and volcanic ashes is the contained vegetable matter. Although the porosity of clay varies from 40 to 70 per cent., the fineness of the grains causes slow percolation.*

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* See also p. 703.

PART II
COLLECTION OF WATER

CHAPTER 5

INTAKES*

Requirements.† Intake protects stream or lake end of conduit from injury by ice, debris, and navigation, and admits water to pumps, pipe line, or aqueduct in best state practicable. To this end it is situated at some distance upstream or to leeward of points of possible pollution, at such depth as to preclude entraining of air, and contains devices for coarse screening, and sometimes equipment for excluding silt.‡ Selection of proper location is important. In small systems, intake pipe may simply end in a screened bell-mouth,

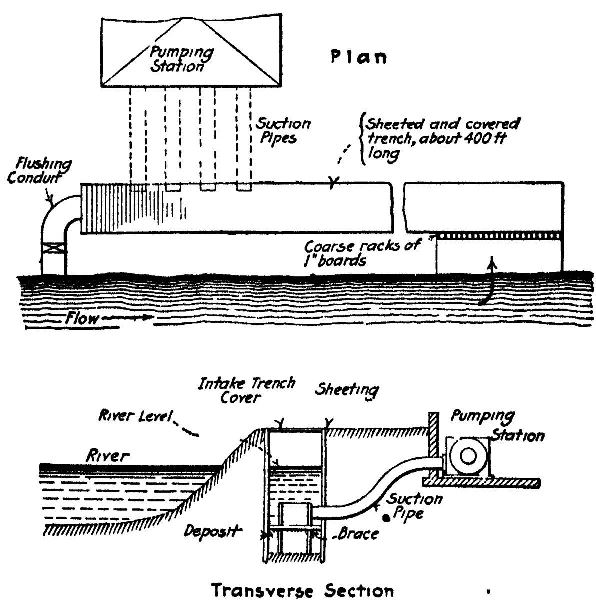

FIG. 18.—River intake. Shore type.

(For smaller intakes, dimensions have been made correspondingly smaller; a 10-in. intake of this type has been built.)

supplied by pipe manufacturers; a foundation is sometimes used to keep end from sinking into silty bottom. On medium-sized systems either an exposed or submerged crib resting on bottom supports and protects the pipe end and

* For "Infiltration Galleries" see Chap 11

† See *Water and Water Eng.* 1923, pp 368 and 411

‡ For examples on small works, see *E R.* Nov 12, 1898, p 515; and Feb. 15, 1896, p. 187. A design used at water stations on the Baltimore & Ohio Railroad is shown in *E. N. R.* Oct. 7, 1920, p. 701.

screens the supply. On large intakes,* such as used for reservoir outlets, or in large rivers, or on the Great Lakes, masonry towers are provided with screened and gated ports at various levels, to take advantage of the varying qualities at different depths. Means must be provided for keeping the screens clean, particularly from leaves in autumn and ice in winter. (See page 83.)

Port areas should be ample to reduce velocities at entrance, so that particles which might deposit in the pipes will settle out. Leonard Metcalf[1] advises limiting velocity to 0.5 ft. per sec. to avoid trouble with fish clogging screens. On large intakes, port areas and screen openings should be at least three times the area of the intake pipe; only the area below water level should be considered in calculations. The Associated Factory Mutual Fire Insurance Company recommends a screen area equal to 1 sq. in. per gal. per min. capacity of pipe. This will approximate an area ten times the suction pipe of small steam pumps; in less turbid water, ratio may be reduced to 5.

A river intake, of excellent type, is shown in Fig. 18. A sheeted and covered trench in the river bank parallel to the flow had cross-sections sufficiently large to give low, depositing velocities, thus permitting grit to settle before reaching the suction pipes. By opening the flushing conduit at suitable times, strong flows were created which removed the sediment.

Reservoir intakes† are often incorporated in the dam and have equipment of the same type as for large intake on rivers and lakes. Blow-offs are provided. Gates are subjected to high heads and should be geared or power-operated. Ports at different elevations are needed to take off the best water during the semiannual overturns. Stop-plank grooves and stop disks are provided to facilitate gate repairs. The gate valves on the effluent should be in pairs so that the downstream one, used for throttling to kill the unrequired head, may be readily removed. The cost of high towers may be eliminated by a screen chamber downstream from the dam. The screens are under pressure; for installation at Wilkes Barre, see *J. A. W. W. A.* Vol. 5, 1918, p. 182. Pressure strainers‡ are common in power house practice; they are so constructed that one compartment may be revolved out of service for cleaning, without stopping flow. In designing intakes, it should be noted that divers require manholes 3 ft. in diameter.

Intake pipes are generally cast iron with flexible joints (see Table 118 page 418), laid at a depth to escape damage. They should be of ample size to reduce friction, but not so large as to cause deposit of sediment. To save pumping charges on long lines, intake lines deliver by gravity to a pump well. Where streams are liable to great fluctuations in level,§ pumps must be placed in a subterranean pit.‖ For surges in intakes see *Can. Eng.*, May 15, 1923, p. 502. Intake capacities should be increased with the consumption. To save cost of new construction at Geneva, Switzerland, a booster pump was employed off shore.[2] Rochester intake is steel pipe. Chicago[4] and Cleveland[5] have large intake tunnels.

* For an extended discussion of large intakes, see WEGMANN'S "Conveyance and Distribution of Water for Water Supply," D. Van Nostrand Company, 1918, p. 218.
† See "High-pressure Reservoir Outlets," by GAYLORD, U. S. Reclamation Service, 1923.
‡ Manufactured by Blackburn-Smith Corporation, N. Y.; and Elliott Twin Strainer, Co., Jeannette, Pa.
§ A floating tube is employed at Steubenville, O.[3]
‖ See example in *E. R.*, Aug. 19, 1916, p. 230; and *E. N. R.*, May 9, 1918, p. 905.

Intakes on the Great Lakes.[8] Experience has demonstrated that, not considering pollution, intake pipes of considerable length are required in lakes to maintain draft, owing to the action of ice and the movements of sand, particularly in shallow waters. Off-shore storms in shallow water erode the sand bottom rapidly, tending to clog intakes or embarrass pumping operations through wear on the pumps. Ice, driven by up-shore winds, piles in huge windrows, often 10 to 20 ft. above the lake level, in shallow water, filling the lake solidly to the bottom to such an extent as to shut off water supplies drawn close to the shore; 2000 to 3000 ft. off shore in 20 to 30 ft. of water is required. In larger supplies, particularly at Chicago, much trouble is experienced with anchor ice as far out as 4 mi. in 40 ft. of water; it is largely a matter of total draft in one locality and the velocity of the water as it enters the intake.

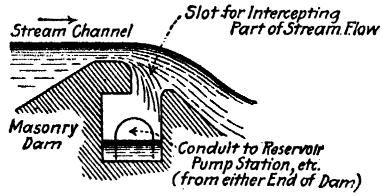

FIG. 19.—Differential intake.[9]

In depths exceeding 40 ft., the influence of waves is insufficient to cause serious washing and consequent turbidity. The distance of the intake from the shore

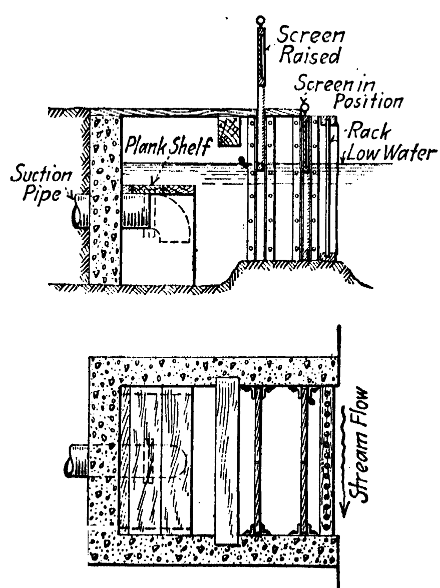

FIG. 20.—Factory intake recommended by Associated Factory Mutual Fire Insurance Company.

also has a bearing. Allowance must be made for deterioration in carrying capacity of the pipes and against the reasonable variations in the rate at which water is demanded. This is particularly important in the smaller supplies.

Anchor ice and frazil* are very important in Great Lakes. They are variously termed frazil, ground ghu, lappered ice (Scotland), moutonne, and spicular ice. Anchor ice is formed on bed; frazil between bottom and surface, and only when there is open water above. On Lake Michigan, further conditions for formation of anchor ice and frazil are westerly wind and thermometer below freezing; never when sun is shining, or when sky is clouded at night. To clear the intake at Lake Forest waterworks, the valves in the well are closed, thereby throwing any pressure remaining in the mains upon the intake through a by-pass. At times, a pressure of 75 lb. has been maintained for 20 min. before there would be noticeable relief. About one out of five times this method, if tried before sunrise, has proved satisfactory; rest of time more water is wasted than comes in through the intake before it is again clogged.[11] Such troubles with ice are experienced on other lakes, also, and along the Mississippi and other rivers. Studies by Murphy in Canada indicate that prevention of formation rather than removal is the desideratum; he recommends[10] steam piping to keep temperature of valves, gates, screens, etc. 0.001° above the freezing point of water. This small increase in temperature prevents ice adhering to structures. At Detroit, a fender deflects ice from the ports.[6] At Moline, Ill., compressed air is used to free screens of ice.[11] An auxiliary submerged intake was installed in St. Louis.[7]

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* See also, Barnes, "Ice Formation." (Wiley, 1906.)

† Also 1924 report of Water Dept.

CHAPTER 6

WATERSHED DEVELOPMENT BY RESERVOIRS

Economic Development. Development consists in construction of works to form impounding reservoirs wherein can be conserved a portion of the flood flows for use in dry periods.* Points to be considered in studying the economic development of a watershed are: (1) yield of area tributary to each possible reservoir; (2) required development of storage in each; (3) storage-elevation curve for each; (4) cost-elevation curve for each; (5) cost-diameter and diameter-gradient-quantity data for all connecting aqueducts;† (6) combined plot of available-storage-cost curves for all reservoirs and their connecting aqueducts; (7) most economical combination of reservoirs within the watershed, and cost-storage curve for this combination for different percentages of total storage on the watershed. (8) Beneficial results, such as flood protection, which may be credited to cost of watershed development. (9) Possible detrimental effect of watershed development upon water supply of lands below reservoir site.

IMPOUNDING RESERVOIRS

Location. The following conditions must be considered: (a) *Geological*,‡ which affect dam location, choice of dam materials, cost of construction, watertightness of reservoir, ground-water storage, and tendency to siltation (see page 106 and Chaps. 7, 8, and 9. (b) *Topographical*, affording a natural basin. The reservoir must be so located that maximum volume of storage is secured with minimum cost of works and least area flooded. Elevations of outlet aqueducts or pipes, and required volume of storage, determine the elevation of the flow line. Paper location on U. S. Geological Survey maps will indicate areas to be surveyed. (c) *Hydrological*. The yield of the tributary watershed must be of such quantity and so distributed in time as to replenish the reservoir with sufficient frequency to supply the demand after deducting evaporation, seepage, and compensation requirements (see "Yield Studies," p. 87).

Size of reservoir should be investigated as to economy by assuming different flow-line elevations and computing all costs—land, structures, "damages." That flow line is the most economical which results in minimum cost per million gallons of useful storage capacity. Requirements as to quantity, however, are sometimes met at least for a period of years by a smaller reservoir than one having lowest unit storage cost. This study ignores hydrological

* For effect of storage on quality, see Chap. 27, p. 633.

† For calculations of diversion conduits, see *E. N. R.*, July 22, 1920, p. 153.

‡ For particulars, see "Engineering Geology," Ries and Watson (Wiley, 1914); "The Principles gineering [illegible]," by Herbert Lapworth, *Proc. Inst. C. E.*, Vol. 173, 1908, p. 298; "Engineer- atskill Water Supply," by C. P. Berkey and J. F. Sanborn, *T. A. S. C. E.*, Vol. 86, 1923, p. 1; "Investigations for Dam and Reservoir Foundations," by C. M. Saville, *E. N. R.*, June 29, 1916, p. 1229; "Geology and the Decatur Dam and Reservoir Project," by M. O. Leighton, *E. N. R.*, Aug. 16, 1923, p. 264.

requirements, and investigations must follow to assure that these are met; otherwise a reservoir might be too large for the tributary area, and demands in dry years would then expose the marginal area below the flow line for longer than 2 years. An objectionable growth of vegetation would result, which would be costly to remove.

Storage Units. Size of reservoir may be expressed in absolute volumes: mil.-gals. for water supply; acre-ft. for irrigation; mil.-cu. ft. for power development; mil.-gals. per sq. mi. of watershed; in days' supply; or in ratio to mean annual flow. Available storage (used in yield studies) is the storage above lowest level at which full flow can be maintained in the effluent conduit.

One Larger vs. Several Small Reservoirs. Even if a number of small reservoirs gives an economy in first cost over one large reservoir, they are not advisable for the following reasons: (1) Small and shallow reservoirs are easily affected by organic growths, producing undesirable tastes and odors. (2) They are sensitive to the influence of vegetation, swampy areas, etc. (3) Water passes through too quickly, giving no time for sterilizing action. Most pathogenic bacteria will die after 2 to 4 weeks in a large storage reservoir. (4) With a large, deep reservoir, water can be drawn from the surface, bottom, or intermediate depths, thus securing good water from one of these levels, when others are unsatisfactory. (5) The quality of the water is improved by sedimentation* and bleaching. (6) The greater depths of a large reservoir, and the greater exposure to the wind, tend to prevent growths of objectionable organisms. On the other hand, the quality may be injured if the reservoir is so large as to be filled only at times of excessively heavy run-off. During dry spells, injurious substances are likely to collect on the margins.

Estimates of reservoir capacities can be made preliminarily, until more exact maps are available, by planimetering the 20-ft. contours on U. S. Geological Survey maps† photostated up to a scale of 1000 ft. to 1 in. These partial capacities between contours should be plotted on a diagram together with a curve of contour areas at each elevation. Do not fail to indicate limits of available capacity. To construct capacity curves of a reservoir in service, C. C. Jacob[1] measures inflow and outflow simultaneously, and change in water stage, thereby securing a volume figure for one zone. Tests at various stages complete the graph. This procedure ignores evaporation losses, but includes ground storage.

Ground Storage. Storage capacity is computed from the surface contours only, whereas water stored in pervious layers tributary to the reservoir also is available. This ground storage may amount to several inches of rainfall or 5 to 20 per cent. of the reservoir capacity.[3a] Its great value lies in exemption from evaporation. Studies of yield should contain estimates of this storage based on subsurface investigations by test pits. Jacob's method[1] of establishing capacity curves determines total storage, rather than capacity of the basin.

Reserve Storage. In operating the Croton reservoirs of New York City's water system the quantity of water in storage is not allowed to become less than a quantity which, added to the lowest run-off recorded for a year, would equal the anticipated consumption for such year.[58] At Plymouth, Mass., charts were prepared to acquaint the public with the state of the reservoirs.[61]

* See Chap. 28, p. 640.

† Horton emphasizes[2] the greater importance on small drainage basins of the occasional errors in government topographic maps.

Largest Reservoirs in the United States. A list compiled by Allen Hazen is given in *E. N. R.*, May 11, 1922, p. 799 and Oct. 5, 1922, p. 576.

Costs of storage per mg. capacity vary widely. Wachusett is given as $36.40, while Cedar Grove (Newark Water Works), is $957.* Costs of storage should include engineering, interest charges during construction, real estate, highway and railway relocation, and other such items in addition to construction expenditures. On some large reservoirs in Eastern United States, costs of land and proceedings for its acquisition represented 26 to 40 per cent. of all costs.

YIELD

Definitions. *Yield* is the collectible portion of the precipitation on a watershed, gathered on the surface or underground. *Safe yield* is the minimum yield recorded or estimated for a given past or future period. *Draft* is the quantity drawn for use. *Drainage area* of a stream at a given point "is the area of the surface of the earth which contributes its run-off to the stream above that point."[4] In some instances, it includes areas outside the surface divide, which are tributary underground.

Data for Calculations. Safe possible yield for water supply cannot be computed until the following data are obtained: (1) Catchment area; (2) rainfall: (*a*) minimum year; (*b*) series of dry years; (*c*) average of a long series of years; (3) ground-water diagram of the stream; (4) available storage capacity on the stream; (5) loss by evaporation and percolation;[2] (6) measurements of actual run-off of the stream.

Swamps. The Committee of N. E. W. W. A., 1914, considered undrained swamps equivalent to 40 per cent. of their areas as water surface and 60 per cent. upland; drained swamps, 30 and 70 per cent., respectively.

Yield studies† *of existing watersheds* have been made by FitzGerald,[8] F. P. Stearns,[9] Hazen[6] (Table 30), Deacon,[10] Committee of N. E. W. W. A.,[11] (Table 29), and others. They reflect differences of judgment and method.

Table 29. Storage Capacity Required to Sustain Constant Daily Draft from 1 Sq. Mi. Containing Various Percentages of Water Surface[8]

(Based on Sudbury Watershed, 1875–1890)

Constant daily draft, hundred thousand gals.	Storage, million gals. per sq. mi.								
	0%	2%	4%	6%	8%	10%	15%	20%	25%
1.0	0.314	1.289	2.656	6.973	10.992	15.012	26.883	40.224	53.565
1.5	3.006	4.711	7.552	11.573	15.592	19.642	32.983	46.324	59.665
2.0	8.797	9.937	12.802	15.666	20.427	25.742	39.083	52.424	65.765
2.5	17.997	20.637	23.502	26.366	29.230	33.338	45.449	58.524	71.865
3.0	28.473	31.337	34.202	37.066	39.930	43.437	54.599	66.702	78.807
3.5	39.173	42.037	44.902	47.766	59.630	54.137	64.812	75.852	87.057
4.0	51.303	52.788	55.602	58.466	61.643	66.050	77.062	88.076	99.089
4.5	63.553	65.038	66.525	69.488	73.893	78.300	89.312	100.970	127.412
5.0	75.803	77.288	79.105	82.131	86.143	90.550	103.474	129.920	156.362
5.5	88.053	89.877	92.905	95.931	98.958	105.987	132.424	158.870	185.312
6.0	100.651	103.677	106.705	113.781	124.357	134.937	.161374	187.820	214.262
6.5	114.451	121.577	132.154	142.731	153.307	163.887	190.324	216.770	250.744
7.0	139.950	150.527	161.104	171.681	182.257	192.837	219.274	265.546	336.044
7.5	168.900	179.477	190.054	200.631	211.207	221.787	280.343	350.846	421.344
8.0	199.106	208.427	219.004	244.445	270.452	297.460	365.643	436.146	406.644
8.5	250.328	276.275	302.240	328.195	354.202	380.557	450.943	521.446	591.944
9.0	334.078	360.025	385.990	411.945	437.952	465.857	536.243	606.746	677.244

* See tables by M. O. Leighton in *E. N.*, May 7, 1908, p. 504; by Sellew, *T. A. S. C. E.*, Vol. 76, 1913, p. 1462; by Schuyler in "Reservoirs for Irrigation, Water Power and Domestic Water Supply," (Wiley, 1908), p. 549; and *Report* by Pittsburgh Flood Comm., 1913.

† See also Chap. 3.

Hydrological studies of safe yield are necessarily based upon practical considerations of rainfall and run-off, or, if these data are not available, upon like data from a similar watershed, properly weighted for local conditions.* In calculating safe yield from large watersheds, it is usual practice to provide for a safe uniform draft for a series of years. A large amount of storage which cannot be drawn off in a single year, and which can be exhausted only after the reservoirs have failed to fill in a wet season, represents a better condition than a reservoir holding but 1 dry year's supply; because (*a*) 2 or more dry years are less likely to occur in succession, although more commonly occurring at intervals; and (*b*) a larger reservoir gives a longer period for action after warning of need of an additional supply. A water famine scare comes long before the reservoir is exhausted to the level assumed in capacity computations. A reservoir which cannot be emptied by the draft in less than 3, 4, or 5 years allows for sufficient warning against the possible shortage, for most communities.

Mass diagram[5] affords the best method of studying behavior of streams, but is not recommended[6] unless monthly stream-flow records for 20 years or more are available.

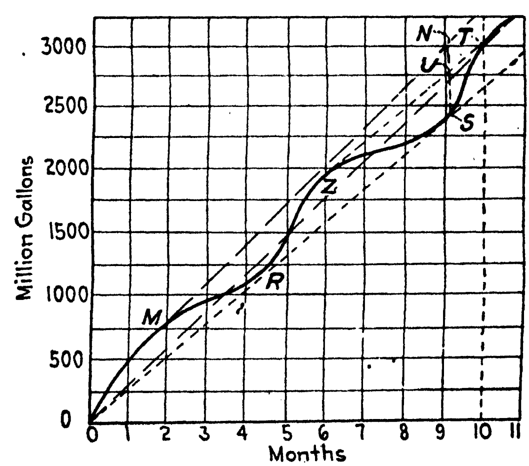


FIG. 21.—Mass diagram.

A mass diagram is constructed by marking off months as abscissas, and, on the ordinates thus located, plotting the summation of flows up to and including that month. If full records are not available, fill in the missing data by the method given on page 3. Units are immaterial—in. of run-off, cfs., or mgd.,—so long as confusion in interpretation is avoided. If records are in cfs., ordinates as read must be multiplied by 60 by 60 by 24 by 30 (= 2,592,000) to convert to total flow in cu. ft. to date.

In Fig. 21, flow for 10 months totals 3000 mg., or at average monthly rate of $3000 \div 10 = 300$, which is represented by dotted line *OT*. Demand is generally represented by a straight line whose slope represents the rate. The line may be broken (change slope) to show different rates for succeeding periods. The demand line may include draft, seepage losses, evaporation, and compensation water. If

* Hazen[70] modifies run-off data for Eastern United States by increasing 0.64 in. per year for each 100 ft. decrease in elevation.

OT represents the demand, obviously there would be periods, *R* and *S*, when the undeveloped supply would not suffice. From various high points on the ascending mass curve, lines with slopes required by demand rates are drawn downward and ordinates intercepted between mass curve and this demand line investigated. The maximum ordinate represents the *available* reservoir capacity required to maintain this draft throughout the period of the record. The horizontal distances between intercepts of said sloping lines represent depletion periods.

If *TZ* represents a proposed rate of draft from a reservoir with volume represented by ordinate *US*, it would be depleted during the period from *Z* to *T*. Line *OS* represents the maximum possible rate of draft; slope of no demand line should be steeper than this line. Mass diagram has the advantage over other studies that the relative periods of filling and depletion are graphically presented.

Studies by A. S. Burgess,[7] N. Y. City Dept. of Water Supply, of mass curves for streams in Northeastern United States for 20-year periods, indicate that a single year may have 40 per cent. of normal yield; his methods of analysis are ultraconservative. Several mass-curve studies are outlined in *E. N.*, 1913, Sept. 11, p. 497, and Dec. 25, p. 1290. For small watersheds, Suter prefers duration* curves to mass curves.

Methods for Short Records. Hazen† applies to short-time records, or to streams with no records, the records of comparable streams by means of a "coefficient of variation." To compute this coefficient, tabulate annual flows in any unit and find the mean. For each year, compute the variation from this mean (ignoring plus and minus signs), and square each difference. Add these squares, and divide sum by one less than the number of terms. The square root of this quotient is the standard variation, which is divided by the mean to get the coefficient of variation‡ (c.v.). The following example is given by Hazen:

Compute storage required to develop 35 mgd. from 48 sq. mi. of mountain country with little ground storage, mean run-off being assumed from data for neighboring areas to be 25 in., and coefficient of variation in mean annual flow to be 0.20. The mean annual flow is 48 × 25 × 17.379 = 20,855 mg., or 57.1 mgd. The required delivery, 35 mgd., is 61 per cent. of the mean flow. As this is more than 50 per cent., Table 31 is used. Under c.v. = 0.20, 60 per cent. and 65 per cent. of mean flow available call for storages of 0.31 and 0.35. By interpolation, 61 per cent. calls for 0.318 of mean annual flow; 0.318 × 20,855 = 6632 mg., the required storage. To this must be added an allowance to cover loss by evaporation (see p. 35). From the same area for a first instalment of 20 mgd., equal to 35 per cent. of the mean flow, Table 30 is used. No ground storage is assumed. Storage required, 0.128 of mean flow, or 2669 mg. An allowance for evaporation must be made.

Tables 30, 31, and 32 represent different degrees of development and ground-water storage, and, in general, apply to the section of United States designated in the title. Table 30 is for partial developments where reservoir refills each winter and ground storage is a more important factor. Tables 31 and 32 are applicable to high and complete developments, where the reservoirs do not refill each winter, and where the coefficient of variation in annual flow is the important factor.

* See p. 42.

† Reproduced by permission from Merriman's "American Civil Engineers Handbook," published by John Wiley and Sons, Inc., 4th ed., 1920, pp. 1195–1199.

‡ See "Records for American streams," in *T. A. S. C. E.*, Vol. 84, 1921, p. 217.

Table 30. Coefficients for Storage

Northeastern States

Per cent. of mean flow used	Storage in terms of mean annual flow: Impervious soils, No ground storage	Average soils, 30 days' ground storage	Deep gravel and sand, 60 days' ground storage	Greatest natural storage, 90 days' ground storage
50	0.229	0.188	0.147	0.106
45	0.192	0.155	0.118	0.081
40	0.159	0.126	0.093	0.060
35	0.128	0.099	0.070	0.042
30	0.098	0.073	0.049	0.024
25	0.072	0.052	0.031	0.010
20	0.048	0.032	0.015	0
15	0.029	0.017	0.004	0
10	0.014	0.006	0	0

Table 31. Coefficients for Storage

Oregon, Washington, and East of Mississippi River

Per cent. of mean flow available	Storage in terms of mean annual flow									Deduction for 30 days' ground storage*
	C.V. = 0.20	C.V. = 0.22	C.V. = 0.24	C.V. = 0.26	C.V. = 0.28	C.V. = 0.30	C.V. = 0.35	C.V. = 0.40	C.V. = 0.45	
95	1.21	1.33	1.46	1.60	1.74	1.90	2.30	2.70	3.10	0.078
90	0.85	0.92	1.00	1.09	1.20	1.31	1.60	1.88	2.20	0.074
85	0.66	0.71	0.77	0.83	0.91	1.00	1.23	1.47	1.70	0.070
80	0.54	0.57	0.61	0.66	0.71	0.78	0.97	1.19	1.39	0.066
75	0.45	0.47	0.50	0.53	0.57	0.62	0.77	0.95	1.13	0.062
70	0.39	0.40	0.41	0.44	0.47	0.50	0.62	0.76	0.92	0.058
65	0.35	0.35	0.35	0.37	0.39	0.41	0.50	0.61	0.74	0.053
60	0.31	0.31	0.31	0.32	0.33	0.34	0.40	0.49	0.60	0.049
55	0.27	0.27	0.27	0.27	0.28	0.28	0.33	0.39	0.49	0.045
50	0.23	0.23	0.23	0.23	0.23	0.24	0.26	0.32	0.39	0.041

* See Tables 30 and 33 for classification and data on ground storage. For larger or smaller amounts the deductions are in proportion to the number of days' storage.

Table 32. Coefficients for Storage

West of Mississippi River, except Oregon and Washington

Per cent. of mean flow available	Storage in terms of mean annual flow								
	C.V. = 0.50	C.V. = 0.60	C.V. = 0.70	C.V. = 0.80	C.V. = 0.90	C.V. = 1.00	C.V. = 1.10	C.V. = 1.20	C.V. = 1.50
90	3.00	3.80	4.70	5.60	6.40	......	......	......	......
85	2.30	3.00	3.70	4.50	5.30	6.10	7.00	......	......
80	1.85	2.40	3.10	3.70	4.40	5.10	5.90	6.70	9.30
75	1.55	2.00	2.60	3.15	3.70	4.40	5.00	5.70	8.10
70	1.28	1.70	2.20	2.70	3.20	3.80	4.40	5.00	7.20
65	1.05	1.44	1.85	2.30	2.85	3.40	3.90	4.50	6.50
60	0.89	1.21	1.60	2.00	2.50	3.00	3.50	4.00	6.00
55	0.74	1.02	1.35	1.75	2.20	2.65	3.10	3.60	5.50
50	0.61	0.86	1.15	1.50	1.90	2.35	2.80	3.25	5.00
45	0.51	0.72	0.98	1.30	1.70	2.10	2.50	2.90	4.40
40	0.42	0.61	0.84	1.12	1.45	1.80	2.15	2.50	3.80
35	0.34	0.51	0.72	0.96	1.22	1.50	1.80	2.15	3.30
30	0.27	0.42	0.61	0.80	1.00	1.25	1.50	1.80	2.75

Table 33. Statistics of Some of the Larger Storage Systems for Municipal Supply in the United States, Jan. 1, 1926†

System	Area in sq. mi. tributary to reservoirs	Mean runoff in in.	Coefficient of variation in annual flows	Storage in terms of mean annual flow	No. of reservoirs	Storage in billions of gal., including reservoirs now building	Actual net available storage in use Jan., 1926
New York City, Croton, Ashokan, Kensico, and Schoharie	968	25.7	0.22	0.65	17	281	261
Boston-Met. W. W	203	21.3	0.24	0.96	9	72	72
San Francisco S. V. W. Co	171	10.0	0.60	2.08	4	62	62
* Denver-Cheesman	1796	1.3	0.58	0.61	1	25	25
Los Angeles (Owens River)	1186	4.8	0.36	0.50	13	50	50
San Diego	219	2.0	1.50	4.00	2	31	15
*Troy, N. Y	67	20.0	0.20	0.51	1	12	12
Hartford, Conn. (Nepaug and East Branch)	93	26.5	0.17	0.29	2	12.5	12.5
Wilkes Barre, Pa	76	20.0	0.20	0.42	10	11.0	11.0
Jersey City, N. J	121	25.0	0.22	0.16	1	8.6	8.6
Bridgeport	86	22.0	0.22	0.24	8	14.0	9.0
Newark	63.7	27.1	0.21	0.38	4	11.4	11.4
*St. Paul	138	4.4	0.45	0.53	7	5.7	5.7
Seattle	135	71.5	0.23	0.06	2	9.4	4.9
Oakland—East Bay Water Co.	77	8.3	0.80	3.03	3	32.6	2.8
Lynn, Mass	....	20.0	0.22	....	..	4.1	4.1
New Haven, Conn	88	22.0	0.22	0.11	..	3.6	3.6
Worcester, Mass	22	22.0	0.20	0.40	10	3.4	3.4
Cambridge, Mass	25	20.0	0.21	0.36	4	3.1	3.1
Springfield, Mass	48	27.0	0.20	0.11	1	2.5	2.5
* Holyoke, Mass	9	27.0	0.21	0.54	4	2.3	2.3

* 1919 data.

N. E. W. W. method (Table 34 and Fig. 22), for computing safe capacities of sources of supply. To apply practically the table or the diagram showing storage capacity required to supply different daily drafts of water from 1 sq. mi. of drainage area, it is necessary to have the following preliminary information:

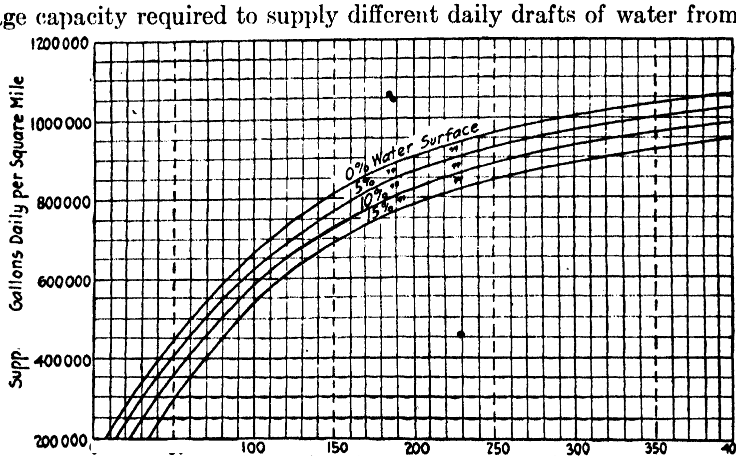


FIG. 22.—Storage capacity required to supply various quantities of water daily from one square mile of drainage area containing various percentages of water surfaces.[11]

Composite diagram based on Abbott Run (R. I.), Sudbury and Manhan Rivers, Tillotson Brook, Wachusett reservoir (Mass.), Naugatuck River (Conn.), Croton River (N. Y.). See Table 34 and Fig. 14.

† Based on Hazen's studies[6] and on correspondence.

(1) Drainage area in square miles. (2) Area of all water surfaces when reservoirs are full, in square miles, together with 40 per cent. of area of undrained swamps and 30 per cent. of area of drained swamps. By dividing total area of water surfaces so determined by number of square miles in drainage area, per cent. of water surfaces when reservoirs are full is obtained. (3) *Available* capacity of storage reservoir, in million gallons. *Available* capacity of storage reservoirs divided by number of square miles in whole drainage area gives storage per square mile.

Table 34. Increment in Storage Capacity Corresponding to Increment in Daily Supply from 1 Sq. Mi. of Drainage Area Containing Various Percentages of Water Surface (Based on Fig. 22)[11]

Increment in daily supply, gallons per sq. mi.	Increment in required storage. Million gallons			
	0%	5%	10%	15%
200,000 to 250,000	7.2	7.2	7.6	7.6
250,000 to 300,000	7.8	7.9	8.2	8.2
300,000 to 350,000	8.4	8.6	8.8	8.8
350,000 to 400,000	9.0	9.3	9.4	9.4
400,000 to 450,000	9.6	10.0	10.0	10.1
450,000 to 500,000	10.2	10.7	10.7	11.0
500,000 to 550,000	10.9	11.5	11.9	12.4
550,000 to 600,000	11.7	12.7	13.6	14.6
600,000 to 650,000	12.9	14.3	15.8	17.6
650,000 to 700,000	14.4	16.3	18.5	21.6
700,000 to 750,000	16.3	18.7	22.0	26.6
750,000 to 800,000	18.8	21.7	26.0	33.0
800,000 to 850,000	22.6	26.5	33.0	43.0
850,000 to 900,000	28.8	35.0	45.0	57.0
900,000 to 950,000	38.7	49.5	62.0	75.0
950,000 to 1,000,000	50.6	71.0	83.0	....
1,000,000 to 1,050,000	74.8	103.0	....	....

Preventing Overflow from Watersheds. Generally the cost is disproportionate to the gain in supply. Studies may be made to determine the additional reservoir capacity required to prevent any flow over the spillway of the only or the lowest reservoir on the watershed as follows: To find the average supply available, plot a mass curve similar to Fig. 21. A straight line *OT* drawn to connect the ends of the curve indicates the average supply obtainable if waste is to be avoided. A line *MN* is drawn parallel to *OT* and tangent to the highest point of the mass curve. The maximum ordinate, *NS*, between the mass curve and line *MN* represents the maximum storage required, if waste is to be minimized.

Average Yields. Experience and computations have shown that only small storage is necessary in New England to obtain 200,000 to 300,000 gal. daily per sq. mi.; but for larger supplies, say 800,000 to 1,000,000 gal. daily, large storage is required for absolute safety in dry years, little of this storage being utilized in ordinary years. As the size of the reservoir is increased beyond moderate limits, a greater proportionate increase in the quantity of storage is required for each additional million gallons per day which such storage will furnish, and it is generally found inexpedient to attempt to secure from a watershed more than 80 per cent. of the average run-off. A general

rule is that above, say, 500,000 or 600,000 gal. daily per sq. mi., each additional 100,000 gal. requires much more additional storage than the preceding 100,000 gal., and a point exists, beyond which, unless storage can be obtained at remarkably low cost, it is not warranted by economics to proceed. Notwithstanding this general rule, there are exceptional cases in which a larger draft may be obtained at moderate cost.

Compensation water is the quantity that must be allowed to pass the diversion works to satisfy holders of prior rights on the stream. It is variously determined in the absence of specific requirements in the grants under which the reservoir is built. For the Ramapo diversion, Bayonne, N. J., it was accepted by the State Department of Conservation and Development of New Jersey as the dry-weather flow of the stream. In England, the requirements are fixed by Parliamentary acts.* All water rights affected should be economically investigated; on large projects, a department is sometimes formed to adjust damage claims.

OVERFLOW WEIRS AND SPILLWAYS†

Requirements. Many earth dams have failed from overtopping when spillways proved too small to carry off floods. Waste weir and overflow channel capacities should be proportioned to greatest possible run-off, unless a given situation justifies a reasonable risk to avoid excessive cost. Depth on crest of weir should be so limited that backwater will not cause claims for damages. In Eastern United States, a waste weir and its channel should be of sufficient dimensions to carry off a 6-in. depth of water on the watershed in 24 hr. In some localities, small reservoirs may require a much more liberal provision. Eddy[71a] says that proportioning on a 6-in. basis has "long been known to be unscientific, and in many cases, has resulted in inadequate provision." Cloudbursts seldom cover an area greater than 40 sq. mi. Meyer computes the area of an excessive rainstorm to have a diameter of 15 mi.[13] On account of the retarding effect of water surfaces, spillways for large reservoirs may be relatively smaller than for small reservoirs. Spillways for Wachusett and Catskill reservoirs were proportioned to discharge 8 in. of run-off per 24 hr. Croton, Sudbury, and the Scioto dam at Columbus were designed for 6-in. run-off. Wanaque overflow weir and channel is proportioned to take a flood occurring once in 1000 years in accordance with Fuller's[14] curves.‡ These curves did not give satisfactory results when applied to the San Antonio flood.[71b] § Strange[15] recommends for Indian conditions, a flood 10 per cent. greater than the maximum in a 25-year record. Miami Conservancy District based design of detention reservoirs on a 10-in. run-off in 3 days (maximum rate = 4.5 in. per 24 hrs.) for watershed areas of 250–270 sq. mi., and 9.5 in., in 3 days (maximum rate = 4 in. per 24 hrs.) for watershed areas between 650 and 780 sq. mi. Stony River dam[60a] (watershed = 11 sq. mi.) is designed to take 12 in. rainfall in 24 hrs.

* It is customary to place, on an authority constructing a reservoir, an obligation to discharge one-third to one-tenth of the average gross yield of the reservoir at a point within 100 yd. of the base of the embankment as compensation to mills and other interested parties within 20 miles downstream from the dam.[57a] For allowances on large projects, see *Eng.*, July 22, 1921, p. 169.

† For design, see p. 146.

‡ See diagram, p. 95.

§ Floods in Cane Creek, N. C.,[60b] North Canadian Creek of Oklahoma City Water Works,[74] and other places, have exceeded Fuller's 1000-yr. curve.

It is not economically feasible to provide for the worst floods* in some regions; Table 35 is compiled as evidence.[66]

Table 35. Excessive Floods vs. Spillway Capacity

Stream	Station or country	Date	Watershed sq. mi.	Equivalent run-off, in. per 24 hrs.	Flood flow cfs., per sq. mi.	Annual rainfall, inches
Catawba	Rockhill, S. C.	May, 1901	2,987	1.0	50	----
Catawba	Not stated	May, 1901	1,535	2.3	62	----
Cane Cr.	Bakersville, N. C.	May, 1901	22	50.0	1,341	----
Elk Horn	Keystone, W. Va.*	June, 1901	44	51.0	1,363	----
Santa Catarina	Monterey, Mex.	Aug., 1909	544	22.0	590	25
Estanzuela	Monterey, Mex.	Aug., 1909	3.5	31.0	825	25
Tansa	India	Not known	52.5	25.0	667	101
Khrishna	India	Not known	345	13.0	343	258
Irawaddy	India	Not known	149,810	0.5	13	368
Ganges	India	Not known	367,970	0.2	5	27
Ohio	Cairo, Ill.	Not known	214,000	0.1	3.3	55
Delaware	Lambertville, N. J.	Not known	6,820	1.3	36	45
Colorado	Austin, Tex.	Not known	37,000	0.1	3.3	24.5
Mississippi	St. Louis	Not known	1,226,000	0.03	0.8	----
Nile	Assuan, Egypt	Not known	1,300,000	0.01	0.35	----

* Average flood flow for 14 hr. = 278 cfs. pr sq. mi.

Location of weir and channel is important, as a prime consideration is removal of water from the foot of the weir as fast as it comes over.† It is especially important to prevent cutting of paving or stream bed near the toe of a waste weir or overfall dam, because the stability of the weir or dam may be threatened in time. Topographical conditions often require a channel at right angles to the weir, which results in the piling up of water in the channel to obtain initial velocity-head for starting it off in a new direction. Careful designing is needed to minimize the depth of water at this point. Destructive velocities often occur in waste channels, and failures of paving are not uncommon.‡ Repairs to the spillway of Gibraltar dam are described in *E. N. R.*, Nov. 9, 1922, p. 798. The hydraulics must be carefully examined to assure "jumps" only at points where no harm would ensue. If a curve occurs in the channel, allowance for banking of water on the outside bank must be made (see page 276). Sometimes, as at Costilla Creek,[17] an outlet tunnel is used to carry off the flood waters; attention must be given to getting water into the conduit; generally the tunnel would be of prohibitive diameter. At Davis Bridge, Vt., a shaft spillway discharges water into a tunnel.[18] At Elephant Butte dam, the spillway channel is a trough 50 ft. wide and 20 to 25 ft. deep, cut out of the flanking hillside and sloping steeply to the stream-bed; capacity, 30,000 cfs.[75] Twin spillways at Lahontan dam converge in a pool; one spillway rests on piers and the earth embankment of the dam.[68] Williams[57b] advocates provision of an auxiliary spillway at a separate location where a low dike can be thrown up to retain the ordinary floods. This dike would be overtopped by floods once in 50 years, and relieve the main spillways. Such an expedient reduces the cost of the spillway.

Waste Weir and Rise of Reservoir.§[19] The following equation gives rise of surface of reservoir under flood conditions (prismatic reservoir, uniform inflow)

* For flood flows, see pp. 62 and 95.
† For hydraulic jump at foot, see *T. A. S. C. E.*, Vol. 80, 1916, p. 390.
‡ See p. 274.
§ See, also, "Determining Regulating Effect of Storage Reservoir," by R. E. Horton, *E. N. R.*, Sept. 5, 1918, p. 455; and studies in *Technical Reports*, Part VII, Miami Conservancy District, 1920.

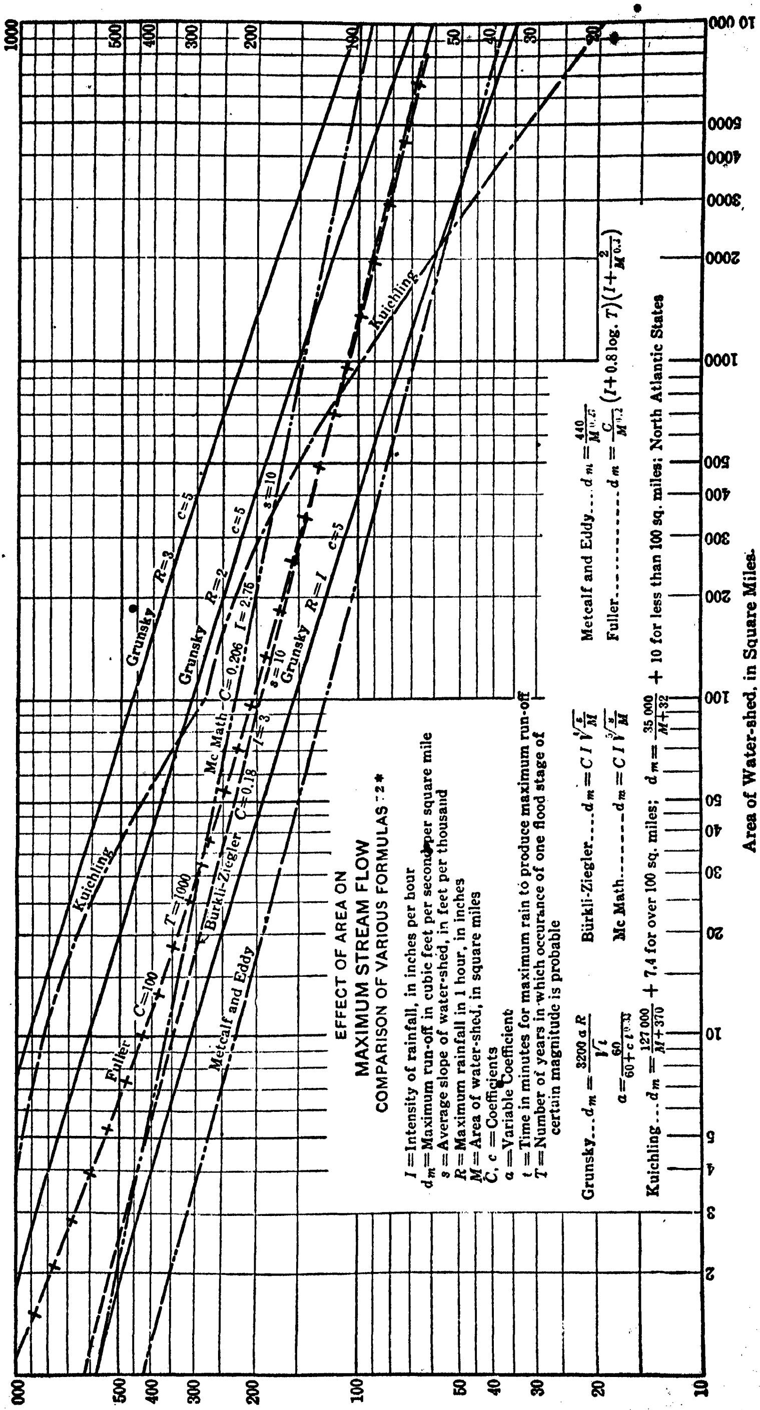


FIG. 23.

* See also p. 61; also Jarvis, Proc. A. S. C. E., Dec., 1924, Plate VIII.

$$t = \frac{A(H_2 - H_1)}{I - \frac{Q_1 + Q_2}{2}}*$$

t = time in sec. for the surface to rise from H_1 to H_2 as observed on the weir.
H_1 = depth in ft. on weir at beginning of flood flow.
A = area of reservoir, sq. ft., at weir level.
I = rate of inflow, cu. ft. per sec.
Q_1 = outflow, cu. ft. per sec., corresponding to H_1.
Q_2 = outflow, cu. ft. per sec., corresponding to H_2.

Table 36. Temporary Storage above Waste Weir Crest, Surface Area of Reservoir, Equivalent Quantity Collected, or Run-off, from Watershed in 24 Hr., and Length of Waste Weir for Three Assumed Reservoirs and Watersheds, D, E, F.

Assumptions:		D	E	F
Area of reservoir, at weir crest level	sq. ft., =	1,000,000	10,000,000	100,000,000
	acres =	23	230	2,300
Area of watershed, sq. mi.	=	1	10	100
1 in. collected from watershed in 24 hr. is equivalent to run-off of 26.888 cfs. per sq. mi.		27	270	2,700
1 in. depth on watershed, cu. ft.		2,323,000	23,232,000	232,320,000
	gal.	17,379,000	173,787,000	1,737,870,000
Length of shore line, or perimeter of reservoir, at crest level = $10\sqrt{A}$	ft.	10,000	31,600	100,000
	mi.	1.9	6.0	19

Average slope of shores 1 vertical on 5 horizontal

A = area of reservoir at level of waste weir crest.
p = perimeter of reservoir at level of waste weir crest.
H = height, or rise, above level of waste weir crest.
G = gain in storage above level of waste weir crest.

Gain in storage, $G = AH + C_wH^2p$ (very nearly) (1)

C_w is a coefficient depending upon slope of shore, and in case assumed $= 2.5$ (*i.e.*, ½ base, or horizontal, of slope ratio, 1 on 5).

Area of water surface for any value of H is:

$A_s = A + 2C_wHp$ (very nearly). (2)

For weir length, broad crest, Q = run-off, cfs. (after steady conditions have been attained), $= 3.40LH^{\frac{3}{2}}$ nearly enough; whence weir length, in ft.,

$L = Q \div 3.40H^{\frac{3}{2}}$. (3)

Formulas, (1), (2), (3), Tables 36 and 37, and Figs. 24–26, are useful for determining storage gained by flashboards or by raising a waste weir or dam; run-off required from watershed to cause such gain; quantity going into temporary storage during a freshet while the head increases on the waste weir; also length of weir for given head H and corresponding assumed conditions.

* Q_1 and Q_2 may be expressed in terms of weir length and head; see graphical solution by J. C. Stevens, *E. N. R.*, Dec. 22, 1921, p. 1031.

Economic Length of Waste Weir. For a given inflow into a reservoir the flood level is a function of the length of the weir. The top elevation of the dam having not yet been fixed economically, and assuming the top of the

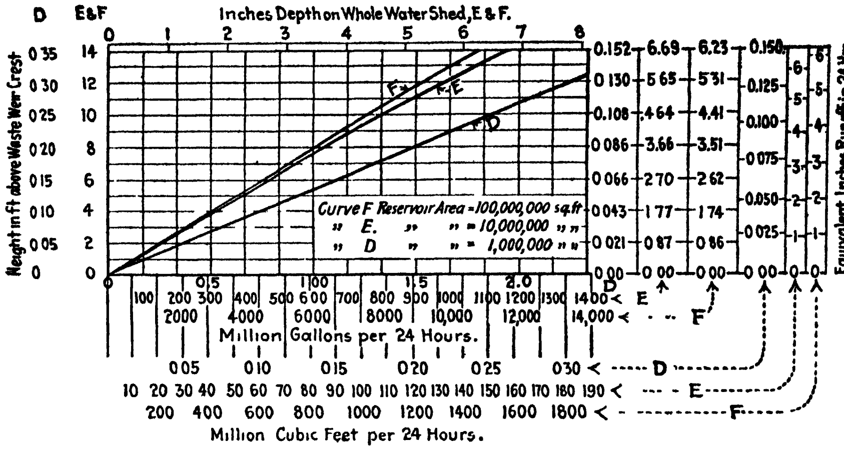


FIG 24 —Quantity collected, or run-off, from watershed in 24 hours.

dam to be at a constant distance above flood level, the inflow provided for will vary with assumed flood level, and consequently with the length of weir. The economical weir length will, therefore, be the one corresponding to the mini-

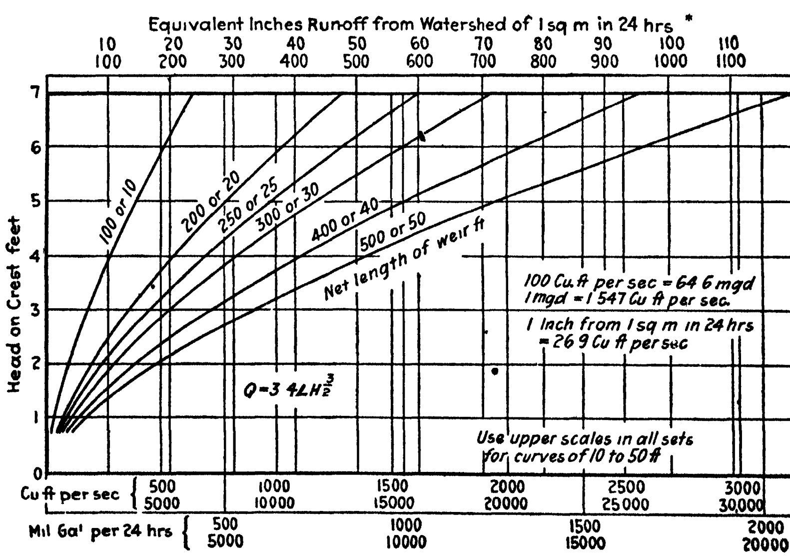
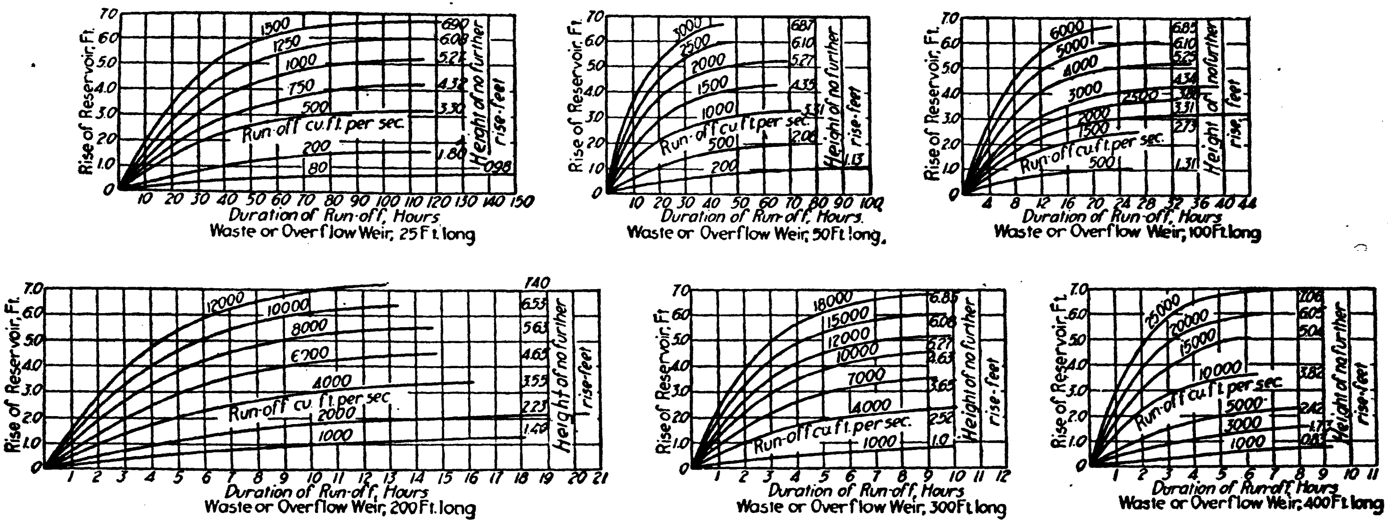


FIG. 25.—Discharge of waste weirs of various lengths.

* These units represent the run off in inches per day × area of watershed in square miles.

mum point on the curve of "height of dam: total cost of reservoir." In plotting this curve, "cost of reservoir" should be plotted as ordinates, that is, to a vertical scale, and "length of weir" as abscissas. Cost of reservoir includes the items enumerated on p. 87.



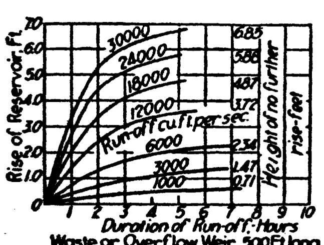


Fig. 26.—Impounding reservoirs, relation of run-off, rise and waste or overflow.

Formula and Notes:

$$t = \frac{5163M}{ab^2}\left[1.151 \log\left(1 + \frac{3abh^{1/2}}{(a - bh^{1/2})^2}\right) + 0.9069 - 0.0302 \tan^{-1} \frac{2bh^{1/2} + a}{1.732a}\right].$$

t = duration of run-off hours.
a = $\sqrt[3]{\text{run-off, cu. ft. per sec.}}$
b = $\sqrt[3]{3.33L}$
L = weir length, feet.
h = rise of reservoir above weir crest, feet.
M = sq. mi. of water surface at crest level.

Formula is in such shape that angle function is read directly from trigonometric tables (*i.e.*, not in π measure).

Run-off, as here used, means strictly the quantity of water entering the reservoir,—evaporation and seepage outward being disregarded.

Height of no further rise is the head on the weir which causes a discharge just equal to the run-off, or inflow.

These diagrams apply to 1 sq. mi. of water surface, and err, on side of safety, in assuming no increase of area with rise. For any other area, multiply duration of run-off in hours by area in sq. mi.

Duration of run-off is measured from time reservoir level reaches weir crest. Rate of run-off is assumed constant.

Table 37. Temporary Storage above Waste Weir Crest,* Area of Water Surface, Corresponding Run-off and Length of Weir

H ft.	*G* million cu. ft.	*A*, million sq. ft.	Inches depth on water-shed per 24 hrs.	Run-off, c.f.s. *Q*	Waste weir, length, ft.
Reservoir D. Area 1,000,000 sq. ft. (23 acres)					
0.05	0.050	1.002	0.021	0.56	14.8
0.10	0.100	1.005	0.043	1.15	10.9
0.15	0.151	1.007	0.066	1.77	9.1
0.20	0.201	1.010	0.086	2.31	7.7
0.25	0.252	1.013	0.108	2.91	6.8
0.35	0.353	1.017	0.15	4.03	5.7
Reservoir E. Area 10,000,000 sq. ft. (230 acres)					
1	10.079	10.188	0.4	11.6	34
2	20.316	10.376	0.9	23.5	24
3	30.711	10.564	1.3	35.6	20
4	41.264	10.752	1.8	47.7	18
5	51.975	10.940	2.2	60.1	16
6	62.844	11.128	2.7	72.8	15
7	73.871	11.316	3.2	85.7	14
8	85.056	11.504	3.7	98.7	13
9	96.399	11.692	4.2	111.8	12
10	107.900	11.880	4.6	125.0	12
11	119.559	12.068	5.1	138.5	11
12	131.376	12.256	5.7	152.3	11
13	143.351	12.444	6.2	166.0	10
14	155.484	12.632	6.7	180.5	10
Reservoir F. Area 100,000,000 sq. ft. (2295 acres)					
1	100.250	100.500	0.4	1,160	342
2	201.000	101.000	0.9	2,320	242
3	302.250	101.500	1.3	3,500	198
4	404.000	102.000	1.7	4,690	172
5	506.250	202.500	2.2	5,880	155
6	609.000	103.000	2.6	7,060	141
7	712.250	103.500	3.1	8,240	131
8	816.000	104.000	3.5	9,470	123
9	920.250	104.500	4.0	10,680	116
10	1,025.000	105.000	4.4	11,880	110
11	1,130.250	105.500	4.9	13,100	105
12	1,236.000	106.000	5.3	14,300	101
13	1,342.250	106.500	5.8	15,550	97
14	1,449.000	107.000	6.2	16,800	94

* For weir tables, see p. 787.

Spillway discharge diagrams indicate: (1) the level to which the reservoir will fill; and (2) the maximum flood which will pass downstream. The general effect of a reservoir is to flatten the peak of a flood hydrograph. Value of retarding reservoirs is discussed in *Technical Report* VII, Miami Conservancy District, 1920, which also contains a rigorous mathematical presentation. See also Scheidenhelm in *T. A. S. C. E.*, Vol. 81, 1917; also *E. N. R.*, Aug. 9, 1923, p. 235.

Macy[20] proposes the following approximate method of calculation:

Data needed: (1) hydrograph showing hourly flows of flood studied; (2) area of reservoir at elevation of weir crest; (3) weir length. Compute data for left-hand curve of Fig. 29 by modified weir formula: Q in billion cu. ft. per hr. = $0.000012\ LH^{\frac{3}{2}}$. Then, storage on crest equals $H \times$ Area from (2). *Assumptions* for Table 38: (*a*) Flow for first hour is all stored on spillway crest, *i.e.*, no overflow, so that zeros are entered in columns IV and V for the first hour. (*b*) The head on the crest at the end of each hour determines the flow rate for the ensuing hour. Solution may be expressed graphically or in tabular form.

Table 38. Calculations for Spillway Discharge

I Hour	II Flood discharge, billion cu. ft. per hr.	III Integrated flood discharge, billion cu. ft.	IV Discharge over spillway, billion cu. ft. per hr.	V Integrated spillway discharge, billion cu. ft.	VI Storage on crest, billion cu. ft.
1	.007	.007	0	0	.007
2	.008	.015	.0001	.0001	.0149
3	.008	.023	.0001	.0002	.0228
4	.009	.032	.00015	.00035	.0316
	etc. to				
54	.336	9.615	.324	5.392	4.223 Max.

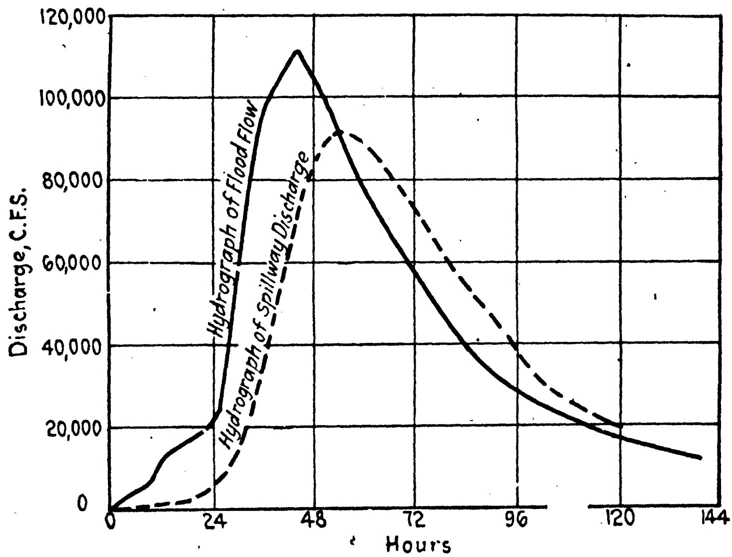


FIG. 27.—Hydrograph of assumed flood run-off.[20]

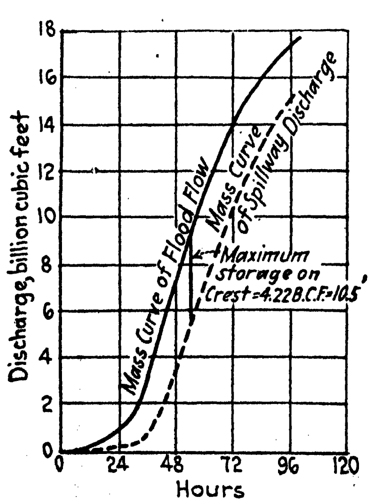


FIG. 28.—Mass curve of flood flow, spillway discharge.[20]

The flows from the hydrograph (Fig. 27), are recorded in column II, which is integrated to form column III, the mass curve data for the flood inflow. Spillway discharge is derived directly from Fig. 29. (It is advisable to plot the lower portion of Fig. 29 on a large scale as the quantities are small.) It is integrated, giving column V, which is the mass curve of the spillway discharge. Column VI is column III less column V. The computation is carried on until the maximum value of spillway discharge is obtained.

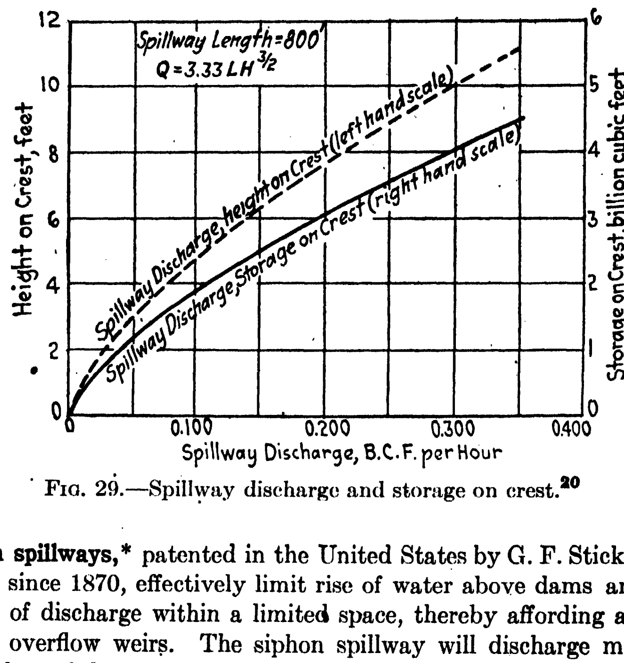


FIG. 29.—Spillway discharge and storage on crest.[20]

Siphon spillways,* patented in the United States by G. F. Stickney,† used in Europe since 1870, effectively limit rise of water above dams and provide high rates of discharge within a limited space, thereby affording advantages over long overflow weirs. The siphon spillway will discharge more water, per linear foot of dam, than any form of waste weir, and is especially effective where there is necessity for restricting the fluctuation of the water surface above the crest of the dam. A single siphon, 5 by 10 ft. in cross-section, put in operation by a rise of 1 ft., acting under a head of 16 ft., will discharge as much water as a waste weir 280 ft. long with a depth of 1 ft. Siphons of large capacity, with throats extending well above the highest water level, if properly designed, will operate when rise is sufficient to produce flow through throat from 3 in. to 1 ft. deep, depth depending on size of siphon. The maximum head that may be utilized for siphonic flow is about 33.9 ft. at sea level, and this decreases approximately 1 ft. for each 850 ft. of altitude. The outer shell of the siphon must withstand the atmospheric pressure, which, under high heads, may amount to 2100 lb. per sq. ft. at the crown, decreasing to 0 at inlet and outlet. While desirable to have the greatest practicable head on spillway, siphons function effectively under low heads. A siphon spillway has operated satisfactorily for several years under a minimum head of 4 ft. at the General Electric Co. plant, Schenectady, N. Y. Tests made to determine the rapidity at the plant of the Tennessee Power Co. on the Ocoee River, Tenn.,

* See also "Siphon Spillways," by G. F. Stickney, *T. A. S. C. E.*, Vol. 85, 1922, p. 1098.
† No. 615,736 filed Mar. 20, 1911.

containing eight siphons, 1 by 8 ft., showed that siphons will flow at full capacity in from 3.3 to 5.5 sec. after the air vents are sealed, the shortest time corresponding to the most rapid rise of the water.

There are more than 45 siphon spillways in the United States; heads range from 4 to 33.5 ft.; discharge capacities, from 70 to 20,000 cfs.[22,23] Many have been built in Europe.[24] In Lagolunga dam, 10 siphons with a capacity of 5250 sec. ft., have given "excellent results for several years."[25]

The discharge of a siphon may be expressed by the following formula:

$$Q = Ca\sqrt{2gH}$$

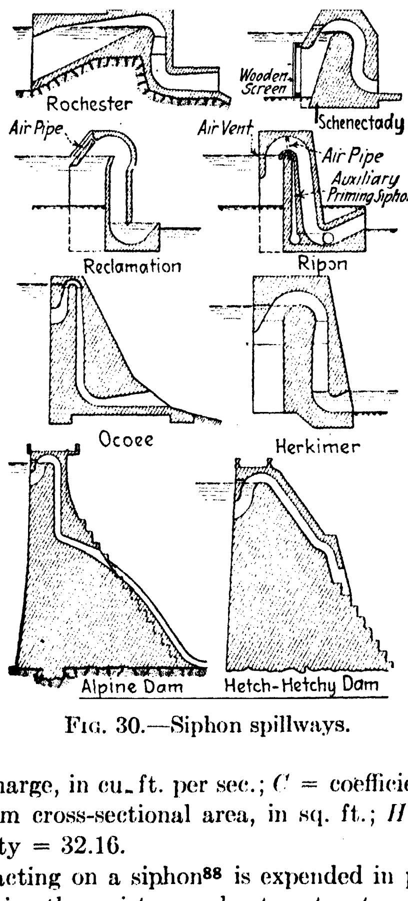


FIG. 30.—Siphon spillways.

in which: Q = discharge, in cu. ft. per sec.; C = coefficient, varying from 0.6 to 0.8; a = minimum cross-sectional area, in sq. ft.; H = head, in ft.; g = acceleration of gravity = 32.16.

The total head acting on a siphon[88] is expended in producing velocity of flow and in overcoming the resistances due to entry, to conversion, to friction, and to bends. This may be expressed by the following formula:

$$H = h_v + h_e + h_c + h_f + h_b$$

in which: h_v = velocity head; h_e = entry head; h_c = conversion head; h_f = friction head; h_b = head lost in bends.

Flashboards* are installed on an overflow dam to prevent waste of water by wave action, or to afford temporary storage above crest level. In flood

* See also p. 155.

time they are removed. Flashboards are removed by hoisting into a car running on flashboard standards (Fig. 31), by supporting pins giving way under stress of flood conditions, or automatically (see Movable Crests). Las Vegas dam, N. M., was designed with water level at the top of the dam, since the reservoir is both fed and drained by conduits, the flow in which can be regulated. To allow for waves, Metcalf and Eddy provided flashboards 2 ft. high, with pins designed to bend under 2 ft. head on the dam crest, in case of a cloudburst, or the failure of the diversion conduit. Since the dam is curved, failure of the pins might cause arch action in flashboards with butt joints, preventing removal until their failure under a higher head. To avoid this, every 50 ft., pins are placed in pairs, which will not fail with adjacent pins. The flashboards are provided with lap joints at these points. These double pins

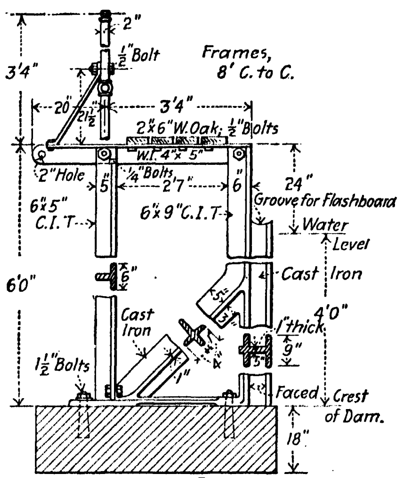


FIG. 31.—Flashboard standards, Muscoot dam, Croton reservoir, N. Y.
(Some recent designs as at Scituate, Providence, and Wanaque reservoirs, Newark, are equipped with light rails, on which runs a gas-propelled car equipped to raise and transport flashboards. See also E. N., Apr. 30, 1914, p. 963.)

remaining steadfast, buckling of the flashboards results, and the water quickly flows off.[28] Brittle pins of high-carbon steel were used at Stony River dam[60c] (U. S. Patent No. 1202228).

Pin, size, and spacing.[30]

$$x = \frac{1,259,340d^3}{S\, lh^2} + \frac{2}{3}h.$$

x = height of water in ft., above dam crest when pins begin to bend.
h = height of flashboards, ft.
l = spacing of pins, ft.
d = diam. of pins, in.
S = ultimate strength of pins, lb. per sq. in., assumed at 69,500 for Wayne iron by the Essex Co., Lawrence, Mass.

Movable crests fulfill the function of flashboards. Many types are automatically movable, to afford a larger waterway for passing floods. Their advantage lies in maintenance of nearly constant water level. There are innumerable forms of wickets and shutters, mostly patented, many of which

are described in "Improvement of Rivers," by Thomas & Watt (John Wiley & Sons, Inc.,).* Operation in cold weather is difficult, and provisions for this were made at Bassano.[26] Another disadvantage is susceptibility to derangement. Driftwood lodged beneath the horizontal rockers of the revolving gates at Austin dam, Tex.,[15,29] and raised the gates from their hinges. Stoney sluice gates, used on Assuan, Gatun, and other dams, are counterbalanced gates traveling in vertical guides and equipped with roller bearings on the ends. At Bassano dam, such gates were counterbalanced to 90 per cent. of the unimmersed weight;[26] the engineers preferred them to Taintor gates as more certain in operation. Automatic flashboards on Tallulah dam are each 28 ft. long and 6 ft. high, hinged at the dam crest. They begin to drop when water level is 3 in. above normal, and come to rest in a horizontal position, at the level of the dam crest, until raised by the receding flood.[77] Stauwerke gates, controlled in the United States by Fargo Eng. Co., operate on the walking-beam principle; they are said to be sensitive to ½-in. rise in water level; among installations may be mentioned dams at Nashua, Iowa,[78] and Chippewa, Wis. Gates of structural steel mounted in the form of a sector of a circle are known as sector gates. The form known as the Taintor revolves upward out of the waterway, by the aid of counterweights. It is not entirely automatic in operation. An automatic type used at Sherburne Lakes dam revolves downward into a pit.[79] Instead of a sector, a complete cylinder is sometimes employed and the structure is known as a roller crest. On the Grand River Project, U. S. Reclamation Service, a diversion weir is so equipped, and will pass 50,000 cfs. without flooding valuable land, less than 1 ft. higher.[80] Operation is not automatic.

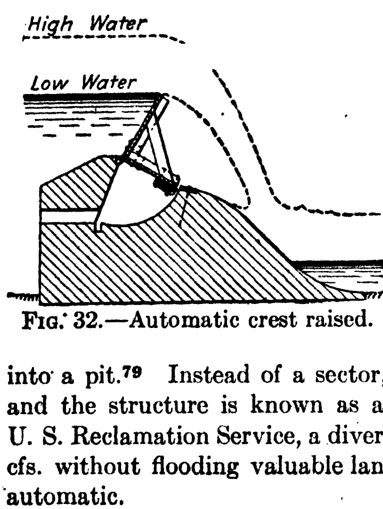


FIG. 32.—Automatic crest raised.

MINOR STRUCTURES

Highway and railroad relocations are necessitated wherever existing locations are to be flooded. It has been found expedient on some projects to enter into a contract with the railroad company to do its own reconstruction. Highway relocation and abandonment require many conferences with local authorities. Economics and local conditions must dictate whether a bridge should be used to carry transverse roads across the reservoir, or traffic should be diverted to a substituted highway around the reservoir. Piers 221 ft. high were required in Dix River reservoir.[81]

Cemeteries occasionally are within the flow line. The bodies must be transferred to a new site. Ashokan, Wachusett, and Providence[64] reservoirs necessitated such work.

Sewers. A reservoir should be located wherever possible so that the population on the tributary watershed is a minimum. Sewers are sometimes required to serve large populations. Jersey City constructed a trunk sewer

* For Stickney crest, see *E. N.*, Feb. 15, 1912, p. 296.

13 mi. long to serve an ultimate population of 48,000 on the watershed above Boonton reservoir (see also Chap. 26).

Intakes and outlet works* serve to introduce reservoir water into the supply conduit, and to restore compensation water to the stream. The controlling conditions for reservoir outlets are the same as for intakes, p. 81, involving screens or trash racks and gates. Provision must be made for taking off large quantities of water at high velocities. High heads complicate the design of sluice gates for low-power operation.† The discs of the outlet valves at Terrace reservoir broke twice in 8 yr.[69] The high velocities‡ on many reclamation projects require the use of cylindrical gates or needle valves.§ Provision must always be made in the head-works for insertion of enough stop planks and stop disks to render possible the replacement of any valve. A rolling curtain is used in Lethbridge Northern Irrigation District.[63] Any valve not replaceable should be of bronze. Valve stems generally are vertical. Inclined stems operate 36-in. gates on Conconully dam. For precautions regarding outlets through earth embankments, see p. 205 and for durability of concrete, see p. 797. For outlet control at Elephant Butte dam, see *E. N.*, Nov. 30, 1916, p. 1015. On the Eastern United States projects, the elevation of the outlet is fixed by the requirements of gravity flow to the point of delivery, but in Western United States and Indian dams, there is the further condition that some volume must be allowed below outlet for silt to occupy. In India, 10 per cent. of volume is allowed for this[31] (see also page 106). In cold climates, uninterrupted operation requires heating of the gate-chambers, as was done at La Loutre dam, St. Maurice River, Que.[82]

Headworks are generally incorporated in the dam, although many earth and rockfill dams have isolated towers in the reservoirs. Ice pressure caused a completely circumferential horizontal crack in the concrete intake chamber of South Fork reservoir, Butte, Mont. Repaired by cement gun.[83]

O'Shaughnessy (*Cal.*) *Dam.* All outlet controls are double, consisting of a slide gate operated by hydraulic cylinders which feeds wells or pipes, from which the water is discharged by a balanced needle valve of Johnson type,‖ with internal screw mechanism for holding the valve absolutely in any intermediate position, and eliminating possible tendency to "hunt" or to close, should any unusual pressure conditions arise in the discharge line. In addition, there is a steel shutter which can be lowered down a special slot and stopped in front of any slide gate, making every valve readily accessible for inspection and repairs. (Information from Mr. M. O'Shaughnessy.)

Fences along aqueducts and other waterworks properties should be substantial but not too conspicuous. Woven wire fence with reinforced concrete posts on 16-ft. centers proved successful on Catskill works.¶ Bids received in 1924 averaged $4 per lin. ft. of fence. Galvanizing should be specified, including ounces per unit of surface (see p. 813). Reservoirs should be protected against trespassers by "non-climbable" fences.

* See 1924 *Report* of Hydraulic Power Committee, *N. E. L. A.*

† See "Some Experiences with Large-capacity Reservoir Outlets," by J. M. Gaylord, *E. N. R.*, Nov. 21, 1918, p. 945, also *E. N. R.* Jan. 17, 1924, p. 127; and "High-pressure Reservoir Outlets," by Gaylord, U. S. Reclamation Service, 1923. 1924 Report on Control and Outlet Works, Hydraulic Power Committee, *N. E. L. A.*

‡ Sixty mi. an hr. (88 ft. per sec.) at Arrowrock is possible, *E. R.* Sept. 30, 1916, p. 409.

§ See p. 453.

‖ See Chap. 20, p. 453.

¶ See WHEELER, E. R. Sept. 18, 1915, p. 361 for methods.

SILTING OF RESERVOIRS

Effect. Filling of reservoirs with silt is serious in Southwestern United States and in other regions having easily eroded soils. It may limit the reservoir's period of usefulness. No location should be finally selected until the possibility of depositions from soils or mine workings has been investigated. The fineness of this silt is such that voids are reduced to 15 per cent.[3b] and capillarity would probably make effective for yield the equivalent of but 5 per cent. voids. Silting not only decreases the available capacity but increases evaporation because of lessened depth of water.

Transportation of sediment[32] takes place by dragging along the bottom, or by carrying in suspension. The weight of particles which will be moved by flowing water is supposed to vary as the sixth power of the velocity.

Table 39. Limiting Bottom Velocity for Stability of Material; Observed by Various Engineers

Material	Velocity, ft. per sec.	Material	Velocity, ft. per sec.
Soft earth....	0.25	Sea pebbles (1.06 in. diam.)...	2.20
Soft clay......	0.50	Brickbats (4.76 cu. in.).......	2.25 to 2.50
Sand.........	1.00	Slate (9.06 cu. in.)...........	2.75 to 3.00
Gravels.......	2.00	Broken stone...............	4.00

Table 40. Descent of Fine Materials through Still Water

Material	Velocity, ft. per sec.
Brick clay, mixed with water and allowed to settle half an hour..	0.009
Fresh-water sand..	0.166
Sea-sand..	0.196
Rounded pebbles (size of peas)............................	1.000

Table 41. Velocities Required to Move Solid Particles in Water

Kind of material	Approximate velocity required on bottom	
	Ft. per sec.	Mi. per hr.
Fine clay and silt..................................	0.25	$\frac{1}{6}$
Fine sand..	0.50	$\frac{1}{3}$
Pebbles, $\frac{1}{2}$ in. in dam	1.00	$\frac{2}{3}$
Pebbles, 1 in. in diam..............................	2.00	$1\frac{1}{3}$

In Eastern United States little silting occurs. No provision is made against it in most waterworks. Most municipal reservoirs have considerable area and depth below the lowest draft line, so a little silting is not noticed. Kensico Lake, N. Y., an 1,800,000,000-gal. reservoir completed in 1884, silted about 3 per cent. in 23 years.[35] Samples taken from four scattered points, Feb. 27, 1907, indicated that the deposit above the original bottom averaged 6 in., (max. 18 in.) partly of organic growth from the overlying water, but principally soil washed in; analyses showed loss on ignition from 6 to 13 per cent., averag-

ing over 10 per cent.[34] Williamsbridge and Jerome Park reservoirs, storing water which has already stood in larger reservoirs, show an accretion of ⅛ to ¼ in. of silt per year. They are concrete lined.[35] Beaver Dam Creek, N. C., brought down 12,000 cu. yd. of sediment in 1 yr. (by cross-sections taken) from a drainage area, bare of forest, and but 14 sq. mi. in extent.[36]

Loch Raven reservoir,[37] Baltimore, with an original capacity of 510 mg., was a long, narrow reservoir, extending between two high hills for 4 mi., in use from 1881 to 1912, filled with silt to such an extent that it had but little larger cross-section than the normal cross-section of the river. By 1900, the capacity of the reservoir had been reduced to 78 mg., although dredging had been carried on from 1896-1900. A city-owned dredge operated from 1900 to 1911, increased capacity to 178 mg. Annual silt deposit 1891–1909 averaged 226,000 cu. yds. The dam was raised 52 ft. in 1923, increasing the capacity to 23,000 M. g. After operating 18 months, silting was reported "too small to be of any moment."

In Western United States allowance must be made for siltation loss, especially on flashy streams, dry a part of the year. Zuni reservoir, N. M.,[3c] lost 47.6 per cent. of capacity in 14 years, so that total life will be about 21 years. A reservoir near Carlsbad, N. M.,[3d] lost more than half its capacity in 15 years. Old Austin reservoir, Colorado River, Tex.,* with initial capacity of 53,490 acre-ft., had lost 62 per cent. in 4 years; flushing by floods improved conditions so that loss in 6.75 years was 52 per cent. The new Austin reservoir silted 84 per cent. in 9 years. J. B. Hawley estimates that in 9 years, Lake Worth reservoir lost 10 per cent.[39] On basis of silt studies, life of Elephant Butte reservoir is forecast as 233 years.[40] A safety factor of 60 years has been provided. La Grange reservoir, Tuolumne River, Cal., lost 50 per cent. capacity in 10 years; deposition represents 0.000067 volume of stream. Analyses of the water show that silt equivalent to 0.000043 volume is carried by the site. Total silt load is 0.00011 volume of the stream.[41] Observed rate of silting in Sweetwater reservoir, Cal., has been 0.5 per cent. per year for the past 20 years.[42] Lake Chabot, Cal., silted 16.5 per cent. in 36 years.[43] Lower Otay reservoir had deposits 5 or 6 ft. deep after 20 years;[44] two reservoirs upstream protected this reservoir.

Prevention of Deposition. Silt originating from smelter wastes, as at Hell Gate dam,[45] are difficult to control. Silt originating from erosion of deforested hillsides might be minimized by reforestation with willows and shrubs, or might be kept from reaching the reservoir by placing simple rock and brush retarding dams in tributary gullies, to check velocities.[3f] Sluices in Assuan dam pass flood waters at times when silt content is greatest, and prolong life of reservoir to this extent.

Removal of Silt. The following methods have been used: (1) Sluicing through a scouring gallery known as a Spanish gate. This method wastes great quantities of water, and removes but a small quantity of silt.[46] Although Austin reservoir was silted 60 per cent. when the dam was washed out, very little sediment was removed. High velocities through the sluice gates of Hell Gate dam,[45] opened to discharge silt, undermined a timber-crib training wall. (2) Hydraulic sluicing or suction dredging is suggested by C. S. Jarvis.[3g] A-

* See also p. 168.

pump is mounted on a barge in reservoir, as at Baltimore's new reservoir. (3) A drainage tunnel extending up the stream bed, with numerous intakes, keeps much silt from reaching downstream end of the reservoir. (4) Reservoir is emptied in winter months for 40 to 60 days, and teams used.[47] (5) Dragline excavators are used at Yuma reservoir, without interfering with water storage.[47] (6) Blasting in time of flood has been suggested.[62]

LEAKAGE*

Besides danger involved in many cases by percolation beneath dams, loss of water from the reservoir is often a serious consideration, and objectionable wetness may be caused in places near the reservoir, leading to claims for damage. Some sites, otherwise suitable, cannot be used because the perviousness of the bottom and sides is so great that objectionable seepage would ensue. Extreme cases are on record wherein reservoirs constructed on such sites have been abandoned, because they could not hold water, without a complete and prohibitively expensive, watertight lining. A 200 mg. reservoir (Blaen-y-Cwm) in Wales, is on the site of an old coal mine.[84]

Limestone regions are liable to be honeycombed with caverns, which would be dangerous on account of leakage. Geologists approved a Porto Rican reservoir on limestone as the heavy clay overlie would clog the pores and cracks of the rock, and because the high percentage of run-off indicated little underground seepage.[85]

Large leakages[48] from Cedar River reservoir necessitated extensive work to lessen them. A clay and gravel blanket was proposed over 200 acres adjacent to the dam but was never executed. 50 acres of a leaky reservoir in northern New York were effectively blanketed with concrete 6 in. thick.[86] Tumalo reservoir due to "lack of essential preliminary engineering and geological studies,"[49] leaked 200 cfs. under 20-ft. head. It is underlain with volcanic *tufa* and lavas, comparatively light in weight and very open-textured. Groundwater level is down 300 to 500 ft., so that leakage out has no resisting head. Leakage under Horse Springs Coulee dam, Whitestone Irrigation Project, is described in *E. N. R.*, 1925, Aug. 27, p. 361; Dec. 24, p. 1046.

CLEARING, GRUBBING, AND STRIPPING RESERVOIRS†

Clearing consists in removal of perishable matter, generally defined to include buildings, or portions thereof of wood, other wooden structures, boards, trees, fences, brush, bushes, vines, shrubs, logs, stumps, roots, grass, weeds, leaves, sawdust, poles, bridges, rubbish, and other organic matter above the surface of the ground, but not sod nor topsoil, although portions of sod and soil may be removed incidentally with other materials. *Grubbing* is removal of all roots greater than a specified size, generally 2 in., which are within a specified depth, commonly 6 in., below the surface of the ground. *Soil-stripping* is shallow excavating of soil containing organic matter, the depth being fixed by the permitted percentage of organic content at the new surface. Clearing and grubbing were done at Ashokan. Several Massachusetts reser-

* See "Watershed Leakage in Gravity Water Supplies," by Horton, *J. N. E. W. W. A.*, Vol. 33, 1919, p. 306.

† See also Chap. 27.

voirs were stripped. Deposits of muck are usually covered with a layer of inorganic earth. Stripping leaves the banks more vulnerable to wave action,* as seen at Wachusett, but, in gravelly or sandy material, stable shores soon establish themselves.

Stumps can be removed: (1) by hand, by 2 men; (2) by burning; (3) by cutting off by patented machine 18 in. below ground;[50] (4) by blasting; (5) by partly excavating in the fall, so that frost action in spring will heave them out; (6) by stump pullers; (7) by pulling over a tree or large stump with tackle, instead of cutting close and leaving low stump to be removed. Explosives should be chosen to produce a slow, heaving action rather than a quick, shattering action. At Kensico reservoir, stumps were cut off close to ground in 1911, when clearing was done, and not removed until 1915; on this account blasting was preferred to other methods.[51] Sixty per cent. dynamite was generally used. Best results were obtained when the ground was frozen.[51] Blasting, as a rule, costs more than stump pullers. Pulling was done at Ashokan reservoir by Holt Tractors.[52]

On cantonment work (during World War), small automobiles (Fords),[53] with cleated wheels in rear, and steam rollers, were used. A method of distilling oils from pine stumps, which destroys the stump far enough below ground to permit plowing over, yielded 17 gal. of heavy oil per stump in Mississippi.[54]

Specifications, Board of Water Supply, New York. Wherever practicable, stumps shall be removed by stump-pulling machines or other similar devices. As stumps, shall be classified the stumps of all trees of whatsoever size, the stumps of all bushes and shrubs, and the stumps of all vines having a woody fiber. Within the areas ordered grubbed, the stones of all stone walls and stone fences shall be widely scattered.

Within the areas ordered to be cleared, all vegetation shall be cut off. Except where a modification is permitted on those areas which are ordered for subsequent grubbing, the heights at which vegetation shall be cut above the ground are as follows: All trees having a diam. of 12 in. or more, 12 in. from the mean surface of the ground, shall be cut off so that the stumps are not more than 12 in. above the mean surface of the ground. All smaller trees and large bushes shall be cut at a height not exceeding 6 in. above the mean surface of the ground. All other vegetation shall be cut close to the ground. All existing stumps not cut under this contract, decayed or decaying stumps mentioned below, standing more than 6 in. above the heights specified shall be cut to heights required for trees. All decayed or decaying stumps, where the decayed portion approximates 50 per cent. of the total original cross-section of the stump, shall be broken off and entirely removed.

Where grubbing is ordered on swamp areas, drainage ditches will be ordered if necessary. Such ditches will be paid for. On land which has been cultivated and on which the principal growth of vegetation is grass or weeds, the time of clearing may be ordered postponed until immediately prior to flooding. All perishable material cut or removed or found on the ground or caused by this work, shall be completely burned, or otherwise satisfactorily disposed of outside of the city land. The masonry portion of any building shall be thrown down and component parts scattered.

* See Wachusett results in *E. N.*, Aug. 13, 1914, p. 344.

Removing Excrementitious Material. Objectional excrementitious material will be ordered removed from privy vaults, barnyards and stables. Such material shall be buried* or otherwise satisfactorily disposed of downstream from the dam or at other acceptable places outside the watershed. After removal of objectionable material, the excavations shall be thoroughly disinfected by application of hypochlorite of calcium. All objectionable material ordered removed shall be transported in vehicles having tight bodies which will not permit scattering of the contents.

The following *unusual points* are also included in these specifications: The entire area reached by the waves is to be cleared. A marginal strip extending vertically from 15 ft. below the normal flow-line to 5 ft. above, is to be grubbed as well as cleared.† Horizontal areas are used in estimating for payment. Much salvable material became the property of the contractor.

Floating Bottom. Peat bogs and the mat of vegetation in swamps tend to float to the surface after being submerged for some time, and often carry stumps with them. This has occurred in a number of places. Stumps tend to become loosened after long submergence. They have little effect on algæ growth; their removal is principally a question of appearance. Floating islands have been formed in this way in Goose Creek reservoir, Charleston, S. C., for example. Stripping of this site would have been so costly that it was decided to let flooding kill the vegetation of the abandoned rice fields. In the process of decay, huge masses were torn from the bottom and floated, and the marsh grass again thrived. Luxurious growths caught the wind and the islands were moved to positions blocking the spillway, causing minor failure of an earth embankment in 1916. These floating islands, 50 acres in extent, also interfered with the beneficent influences of sunlight and aeration. In 1921, they were removed, and 75 additional acres of bottom were cleared, using lighters, at a cost of $54 per acre. There resulted "a marked increase in dissolved oxygen and a corresponding decrease in carbonic acid content."[55] Vegetable growths reduced area of Krian Irrigation reservoir 20 per cent. in 11 years.[67]

FORESTRY AND GRASSING‡

Reforestation of Reservoir Margins. Tracts around large reservoirs, if left to Nature, are of no economic value, and, if not properly controlled, will become detrimental to the quality of water. Such tracts cannot be put under cultivation or used for grazing because the manurial conditions would be objectionable. Forestation of marginal lands protects the water by retarding run-off and decreasing turbidity, prevents wash-outs and silting, hinders free access of trespassers to streams and reservoirs, and supplies marketable timber. Forests guard against floods, winds, snow slides, and erosion, and have great influence over the temperature of the surrounding country. Leaves contain 50 to 75 per cent. moisture. More heat is required to raise the temperature of a pound of water than almost any other substance; hence, leaves

* Schoharie reservoir specifications require burning.

† Schoharie specifications: Areas above contour elevation 1050 (flow line at elevation 1135), where slopes are generally 1 on 4 or steeper, and also some excessively steep slopes above elevation 1135 are to be grubbed as well as cleared.

‡ See also, "Reforestation of Watersheds for Domestic Supplies," by F. W. Rane, Mass. State Forester; "Practical Forestry for Water Works," by E. S. Bryant, *J. N. E. W. W. A.*, 1911; and Collingwood in *Am. City*, February, 1924, p. 147.

absorb more heat than bare rocks or soil. Leaves also tend to keep the air cool by transpiration.* Forests have high influence on evaporation. Dr. Ebermayer, German forest meteorologist, showed that evaporation from forest soil without a layer of mold (humus) was 47 per cent. of that from soil in the open, while with a layer of good mold, it was 22 per cent. When rain falls upon a dense forest, from less than 0.1 to 0.25 of it is caught by the trees, the greater part of this being evaporated therefrom.

Selection of Trees.† Conifers provide the best cover for watershed areas. An evergreen forest presents a pleasing picture at all seasons; its mat of needles keeps frost out of the ground and permits penetration of rain or melting snow. At margins of reservoirs, also, conifers are preferred, as they shed no large leaves to clog outlet screens. *Arbor vitae* or white cedars are good. Trees best adapted for forestation in Northern and Eastern United States are red and Scotch pine, European larch, Norway spruce, Colorado blue spruce, hemlock, red oak, and sugar maple.

White pine‡ has always been a favorite, as it is easy to grow and has a good market value. In common with other five-leaved pines,§ however, it is liable to "blister rust" to which the currant bush acts as host; some states now prohibit its propagation in designated fruiting currant districts. White pine grows faster than red pine, but the latter is recommended by Hawley[87] for watersheds because of freedom from insect attacks.

Best trees for ornamental planting are: for avenues—Norway maple, pin oak, English linden, oriental plane, elm, and sugar maple; for screens—white pine, Norway pine, Scotch pine, Austrian pine, hemlock, white spruce, Norway spruce, Douglas spruce, North Carolina poplar, Lombardy poplar, willow and white birch; for settings for buildings—red cedar, hemlock, Lombardy poplar, Norway spruce, white spruce, larch, beech, oak, tulip and maple. For park purposes—various shade trees should be used and evergreens named above. For shrubs, preference should be given to those having berries, colored barks, and fine foliage rather than floral display, although beautiful flowers when combined with other good qualities are desirable.

Field Work. When buying trees and seeds, American species should always be given preference, as foreign stock has brought diseases and insects of most damaging kinds. The long duration of travel is hard on the stock. The most durable kinds of wild stock to transplant are white and red pine, spruce, hemlock, balsam, oaks, hickory, maple, and basswood. Transplanting of conifers should begin as soon as frost is out of the ground, and may continue until the buds begin to swell. Conifers should not be more than 2 years old for economical handling, as they have long, straggling roots. Seed beds should be enriched by good fertilizer, well-rotted barn manure being the best. Compost heaps of manure and earth should be made each year, which will rot, without leaching, 3 years before using. The addition of unleached hardwood ashes is always valuable. Conifers on reservoir margins should be spaced 6 ft. apart each way, requiring 1250 per acre. Fire guards (unplanted strips)

* Defined on p. 36.
† See "Reforesting Water Supply Land on Catskill System," F. F. Moore, *E. N. R.*, July 12, 1917, p. 59.
‡ P. Strobus.
§ Five-leaved pines include: P. Strobus, P. excelsa (Bhotan pine), P. Lambertiana (sugar or giant pine), P. Cembra, and P. occidentalis.

should be at least 30 ft. wide. Deciduous trees should be planted 50 ft. back of the conifers if desired for landscape effects. Seedlings are planted in seed beds, about three seeds to the sq. in. At the end of 1 year, transplanting takes place. Seedlings remain 2 years in the transplant bed, then they are ready to set out; 30,000 seeds planted in a nursery should produce 10,000 trees. At Wachusett reservoir, the average cost of planting 3 year old white pine was $5.25 per 1000 trees. At Kensico reservoir, the cost ranged from $30 to $60; at Glens Falls,[59] $2.50 to $8.00. In many states, the departments of forestry maintain nurseries where stock can be purchased at cost.

Economics of Forestry. Pine trees 50 years old will be at least 14 in. in diam. and 60 ft. high. It is profitable to cut trees at 40 years, but the value of the increment makes it worth while to wait. At 50 years, white pine will have a value of 46 M ft. BM. per acre. If the stumpage value be taken nominally at $20 per M, the value per acre would be $920. A planted forest needs to be thinned at 15 and 30 years.

Grassing is essential in some situations to prevent sloughing of fine soils on side hills. Waves may attack such hillsides (see p. 199). Hillsides of reservoir on North Canadian River, Okla., were attacked by waves*, at the first filling, and water level had to be lowered and gunite slabs put in for protection.[56]

Grassing Cuts. For grassing cuts and fills along highways, etc., use sand clover as well as white clover in sandy places. Honeysuckle, ivy, and some other vines for slopes, require less care than grass, and permit less growth of weeds.

Grassing Embankments. For binding embankments, where firm turf is required, couch or witch grass is adapted to Northern and Middle States; there are a number of forms, all good hay grasses, objectionable in fallow lands owing to rootstocks. Couch grass makes very tough sod, especially valuable in binding canal banks. Hungarian brome forms a good turf, but the creeping rhizomes (rootstocks) are not as strong as in couch grass. In Southern States, Johnson or finer Bermuda grasses succeed better than couch grass. Johnson grass produces a mass of widely creeping, strong rhizomes, which are hard to eradicate after they have once filled the ground. Bermuda grass does not penetrate as deeply as Johnson; in sandy soils it succeeds better, is of great value as binder on sandy levees, and one of best pasture grasses of the South. If land is moist and soil clayey, knot grass may be substituted for Bermuda. It is not injured by moderate submersion; it is useful as covering for silty deposits on banks. The best method of starting a growth is transplanting rootstocks. The following specification is from John H. Gregory: Surface shall be carefully prepared and raked over, then use at least 50 lb. of grass seed per acre, mixture consisting of 4 parts of beach grass, 4 parts of Hungarian brome, 4 parts of orchard grass, and 1 part of white clover, together with not less than 400 lb. of raw-bone fertilizer per acre; roll well. Embankments may often be protected by sodding a portion of the area. A levee specification reads: Entire surface shall be planted with living sods of Bermuda

* See also "A Phenomenal Land Slide," by D. D. Clarke, *T. A. S. C. E.*, Vol. 53, 1904, p. 322; and Vol. 82, 1918, p. 767.

grass not less than 4 in. sq., placed not more than 2 ft. apart. Sods shall be covered with 1 or 2 in. of earth, as directed.[65]

Miami Conservancy District.* On slopes of dams, which consist mostly of gravel, experiments were made with many varieties of vegetation: brome grass, timothy, red top, Canada blue grass, Kentucky blue grass, sweet clover, Japan clover, wild bunch grass and others, planted in various combinations with or without nurse crops and at different seasons. There were also plantings of honeysuckle vines, roses, horse tail ferns, Boston ivy, sumach and St. John's wort. Up to the present time, sweet clover† and alfalfa have proved to be the best cover for the gravel slopes that can be obtained quickly. Honeysuckle has done very well where planted with a shovelful of topsoil to each plant, but has not yet made a dense cover such as is required to prevent erosion entirely. All of the dams now (December, 1925), have a fairly good stand of sweet clover with a mixture of alfalfa, timothy, and other grasses in various proportions. It is believed that in the course of a few years the soil will be enriched sufficiently so that a sod may be obtained. Based on results from experimental plots, seedings in 1926 will be a mixture of brome grass, fescue grasses and alfalfa, with the possible addition of some blue grass.

Matrimony vine[57c] *(lycium vulgare)* has been successfully used for bank protection along the Susquehanna River at Harrisburg, Pa., and also in protecting the slopes of the Wildwood reservoir embankment, near Harrisburg. The shoots of this vine are planted, and the vine grows along the ground sending out roots, which enter the ground for some depth and then start other shoots. The vine, which was brought from the Mediterranean, is very hardy. In a few seasons, the bank presents the appearance of a continuous tangle of vines, and the topsoil becomes a tangle of roots. Banks thus protected have successfully withstood occasional wave and stream action, but, of course, could not be depended on to resist continuous wave action.

Catskill Waterworks. On portions of Catskill aqueduct embankments and slopes of cuts, and on earth dikes, the following grass seed mixtures were used:

For tunnel portal cuts and adjacent embankments: 25 lb. red top; 5 lb. Rhode Island bent; 5 lb. Kentucky blue; and 17 lb. of timothy per acre.

For rock débris embankments, containing much fine material, and for borrow pits or spoil banks, 20 lb. per acre, 10 lb. each of trifolium incarnatum (crimson clover), and Bokhara clover (sweet clover).

For top soil, 69 lb. per acre: 12 lb. agrostis stolonifera (creeping bent); 9 lb. agrostis vulgaris (red top); 3 lb. bromus pratensis (meadow brome grass); 10 lb. festula ovina (sheep's fescue); 15 lb. lolium perenne (perennial rye grass); 5 lb. trifolium repens‡ (white clover); 10 lb. avena elatior (tall rye oat grass); 5 lb. trifolium incarnatum (crimson clover).

For clayey embankments, use 65 lb. per acre: 8 lb. agrostis stolonifera (creeping bent); 8 lb. agrostis vulgaris (red top); 4 lb. anthyllis vulneraria (kidney vetch sand clover); 11 lb. avena elatior (tall meadow oat grass); 8 lb. fetucca durinsula (hardy fescue); 11 lb. poa compressa (Canadian blue grass); 8 lb. poa pratensis (Kentucky blue grass); 2 lb. trifolium repens (white clover); 5 lb. trifolium incarnatum (crimson clover).

* See report of Miami tests on growing vegetation on gravel dams, E. N. R., July 8, 1926, p. 55.
† A biennial, dying at end of second summer.
‡ Called also Trefoil.

For sandy dry banks, 60 lb. per acre: 10 lb. creeping bent; 4 lb. kidney vetch; 10 lb. tall meadow oat grass; 10 lb. smooth brome grass; 13 lb. sheep's fescue; 13 lb. creeping fescue.

To the above, 10 lb. of oats were added for spring seeding; 10 lb. of rye for fall. The seed expert at Vaughan's Seed Store, New York City, recommended (summer of 1912) for ordinary sowing, at 35 lb. per acre, the following mixture:

25 lb. Italian rye grass (Lolium italicum)	at $0.085	$2.13
12 lb. timothy (phleum pratense)	0.20	2.40
45 lb. red top, unhulled (Agrostis vulgaris)	0.12	5.40
15 lb. Canada blue (poa compressa)	0.19	2.85
10 lb. creeping bent (Agrostis stolonifera)	0.25	2.50
3 lb. white clover	0.40	1.20
110 lb. total		$16.48

All seed must be fresh and good. Crops of some years of a given variety are not so good as others. Test seeds by forced germination of samples in moist sand. Usually it is prudent to select varieties of seeds commonly sold; they are more likely to be fresh. It is much more difficult to start a good growth on an embankment with the usual steep slopes, because the fresh surface is eroded by rains, and moisture rapidly drains away; to counteract this, tops of Catskill aqueduct embankments were dished slightly in some places; but if the top edges of the embankments were made so high that the top did not become level and the character of the earth was such as to retain water, the standing water injured the grass.

Examination of the grass growing on embankments led to the suggestion of a simpler mixture, containing two-thirds red top seed, and one-third perennial rye grass, with a small addition of white clover. Red top of good vitality should give satisfactory germination in soils ranging from moist, rich loam to relatively dry clay. For places without topsoil, red and Bokhara clover may succeed; the function of the red clover (which winter kills), is to grow up and form a root substance, which may serve as food for the second season's growth of Bokhara. Sow in late March or early April, at the rate of 10 to 15 lb. per acre.

Prof. E. G. Montgomery, New York State College of Agriculture, Cornell University, recommends: (*a*) for topsoil of fair quality, 4 or 5 in. thick, red top (hulled) 8 lb. per acre; Canada blue grass, 15 lb. per acre; Kentucky blue, 15 lb. (*b*) For fairly good topsoil, but liable to wash, add 10 lb. of annual Italian rye grass, which persists 2 years, forming a sod. (*c*) For very thin topsoil, sow quack grass, but only as a last resort, as it has no value for hay, and is a menace to neighboring hay crops. Alfalfa is not likely to prove of value as it is expensive to seed, does not form good sod quickly, and dies out unless cut regularly.

Relation of Water-table to Vegetation. Investigations have shown that when the ground-water surface is 5 ft. or more below the surface of coarse soils, no moisture reaches the surface or the roots of vegetation through capillary attraction.

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CHAPTER 7

MASONRY DAMS

Types of Dams and Adaptability. Large dams are generally built of earth, rock fill, or masonry, including reinforced concrete. Choice depends on character of site, availability of materials, cost, life required of the dam, and on degree of security desired. Selection is also influenced by the requirement for overflow, rock fill and earth obviously being unsuitable. Earth dams (Chap. 9), are cheaper than gravity masonry dams where impervious earthy materials are available (Cf. p. 193). Rock-fill dams (Chap. 8) are used where suitable earth is not available, and costs of materials imported for a masonry dam would be exorbitant. Timber dams are employed only for camps or small works; see Fortier: "Timber Dams," *Bull.* 249, Office of Experiment Stations, 1912.

Types of Masonry Dams. Masonry dams may be built of laid stone, cyclopean concrete, concrete, or reinforced concrete. Although gravity is an important factor in the stability of most dams, usage limits the term to solid dams which resist displacement by virtue of great weight. Where topography permits a separate spillway, a gravity dam is designed of the non-overflow type, representative sections for which are shown on p. 175 to 187. If the dam is to take overflow, various hydraulic conditions are to be met (p. 146). Arch dams (p. 135), resist displacement by utilizing the hillsides as abutments. Gravity dams are often curved in plan to secure additional stability* or to fit the rock foundation. Hollow dams resist displacement by their weight, combined with that of the water on the inclined upstream face. The flat-slab type (Ambursen) employs slabs supported on buttresses, while the multiple-arch dam has inclined arches sprung between buttresses, the buttress reaction in both types being downward and forward.

Composite dams are often the most economical. Olive Bridge dam (Ashokan) consists of a non-overflow gravity section, flanked by earth embankments containing core-walls. The East Canyon Creek dam consists of an arch in the valley flanked on one side by a hollow-deck dam, and on the other, by a gravity dam of non-overflow type.[1] The Boonton dam is a gravity dam, which for part of its length acts as a spillway. The Bassano dam is earth, with an Ambursen-type spillway.[62]

Choice of masonry types requires careful weighing of many factors, involving both economics and public opinion. The public still looks upon the solid gravity type as the safest for very high and long dams. Good, impervious foundations are indispensable to solid gravity dams; the only exceptions are low barrages on porous foundations (see p. 149) which are a distinct type. They are overflow and diversion weirs; their height seldom exceeds 20 ft. Hollow dams (see p. 141) may also be used with discretion on porous foun-

* This claim is refuted in slightly arched dams, by Jakobsen, *E. N. R.* June 12, 1924, p. 1020.

dations. Jorgensen[2a] points out that for "reservoir-full" conditions, the material in a gravity dam is worked only to about 50 per cent. efficiency, as compared to a hollow dam. Choice between curved and straight plans depends chiefly upon the character of the site; a narrow gorge with abrupt rock sides naturally suggests an arched dam, but on many sites an arch-shaped dam not only would possess no structural advantages, but would cost more and in some cases might be less secure because not fitting the foundations so well. See p. 126 for treatment of foundations and prevention of flow beneath dams. For merits of other types see: earth dams, p. 193; rock-fill, p. 189; arch, p. 135; multiple-arch, p. 142; flat slab, p. 145.

Failure of dams* is an important consideration to the public, and is the argument that often defeats an engineering proposition near a populous region. Many failures may be laid to inadequate outlay for engineering advice and for preliminary investigations, including the geology, and materials available. Many well-designed structures have been built without engineering supervision due to the owner's false economy in dispensing with engineering advice too soon. A few dams built under engineering supervision have failed disastrously, notably Bouzey dam in France (see also p. 119).

Ice thrust is a force to be considered. It has often wrought injury to dams, of both solid and hollow types.[78]

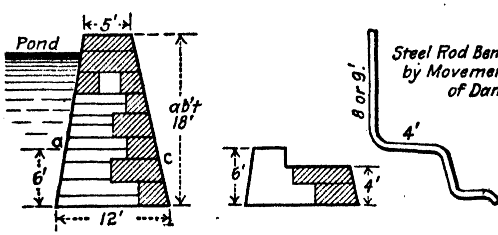


FIG. 33.—Damage by ice to dam at Minneapolis.

At Waldron, Ill., a new dam was built, about 50 ft. below an old one, the old dam not being disturbed. Much trouble has been experienced on the old dam from ice thrust, but after building the new dam, the water passing over the old dam set up such a current that no ice could form between them. This indicates a way to prevent ice from injuring a dam.

At Minneapolis, the minimum temperature Feb. 4 to 15 inclusive, 1899, during injury to a dam, was: Feb. 4, −14°F.; Feb. 5, −15°F; Feb. 6, −14°F.; Feb. 7, −17°F.; Feb. 8, −29.5°F; Feb. 9, −33.5°F.; Feb. 10, −22.5°F.; Feb. 11, −31°F.; Feb. 12, −20°F.; Feb. 13, −8°F.; Feb. 14, + 12°F.; Feb. 15, + 17°F. The damage was noted on the fifteenth. Joints were not continuous from back to front at the plane of fracture. This dam was about 18 ft. high at the break and the lower 4 or 5 ft. were not displaced. The dam failed by being weakened along plane *ac* and by combination of sliding and revolving about point *c*. The unmoved, stepped part extended 200 ft., entire length of opening. The lowest course rested on ledge; dowel pins were freely used; laid in Portland cement. Firm, hard, clear ice, little snow on surface, free from cracks. The resident engineer thought failure was not

* See "Record of 100 Failures," by Jorgensen, *J. Electricity*, Mar. 15, and Apr. 1, 1920, pp. 274, 320.

due to expansion of the ice, but rather to the depressed surface becoming level, as the water rose in the pond on Sundays, the action being on the toggle joint principle. Large temperature changes recorded, however, cannot be wholly disregarded. This dam was used only as an impounding dam, not an overfall dam; there was always 7 to 8 ft. of water on the downstream side.[5]

Magnitude of Ice Thrust. Ice has about eight times as great capability for thermal expansion as steel. This causes the ice to travel up shores of ponds, and to exert tremendous pressures if confined. Pressure of an ice field is augmented by wind influence and by cakes of ice held by the spillway piers. Ice may expand 0.00025 to 0.00066 ft. per lin. ft. of reach. The modulus of elasticity is 180,000 to 360,000 lb. per sq. in. The force considered is the ultimate crushing strength of a field of maximum thickness, placed by various authorities at 12 to 25 tons per sq. ft. Ice thicknesses are recorded in *Monthly Weather Review.* Values for ice thrust used in design is a matter of judgment (see Table 50). Ice pressure is assumed applied at spillway level; the maximum effect on designs occurs in low dams, and the upper portions of high dams. Ignore ice pressure where the mean January temperature is above 40°F. None was considered in designing Boonton or New Croton dams; 17 tons per lin. ft. was used at Columbus,* and 23.5 at Wachusett, Ashokan, and Kensico. For Cross River dam, 12 tons was used, while the same designer adopted 15 tons for Croton Falls dam on account of the configuration of the reservoir, which made larger thrusts probable, and the larger quantity of water impounded. A thrust of 25 tons was used for La Loutre dam, St. Maurice River, Que.[6]

SOLID GRAVITY DAMS†

Requisites. First is ample strength; second, materials should be arranged to furnish this strength with minimum cost. Requisite strength, or margin of safety, and permissible cost will vary widely, from the principal dam of a great reservoir for a metropolitan water supply, situated upstream from populous and expensive territory,‡ to an unimportant power dam in the wilderness It must be borne in mind that worst conditions—conditions which are possible and for which a designer should provide—may not occur for a great many years. That a structure has stood a number of years does not necessarily prove that it is permanently secure.

For conservative practice: (1) Explore very thoroughly all subsurface conditions by test pits and borings, under guidance of a geologist.§ (2) Provide for upward hydrostatic pressure, ignoring the possibility that such pressure may be avoided or reduced by cut-off, coating upstream face, grouting seams, or other means (applies especially to a site where the rock has horizontal seams). (3) Assume that a few feet of thickness of the downstream face of the dam will be ineffective, in some climates, to resist compressive stresses in cold weather. (4) Make due allowance for ice pressures in cold climates. (5) In computations, make conservative assumption as to unit weight of the

* O'Shaughnessy dam.
† For general list of symbols, see p. 128.
‡ For architectural treatment, see Flinn, *E. N.*, Sept. 2, 1915, p. 433.
§ Consult "The Principles of Engineering Geology," by Herbert Lapworth, *Proc. Inst. C. E.*, Vol. 173, 1908, p. 298; "Engineering Geology of the Catskill Water Supply," by C. P. Berkey and J. F. Sanborn, *T. A. S. C. E.*, Vol. 86, 1923, p. 1; and "Investigations for Dam and Reservoir Foundations," *E. N. R.*, June 29, 1916, p. 1229.

masonry. (6) Provide for vacuum under the overfalling water sheet in overflow dams. (7) Carry foundations deep enough to insure stability. (8) If the dam is critically located in respect to a community, allow an ample factor of safety. (9) Wind pressure is neglected.*

Upward Water Pressure. Few masonry dams are free of seepage on the downstream face, particularly where one day's work joins another.† This would indicate that water exists under pressure in the interior of the mass at the joints, where it makes less effective the frictional resistance of the joints, and promotes sliding. Failure of the dam at Austin, Tex., is charged to the action of uplift. Although high masonry dams‡ on good foundations designed with no consideration of uplift have stood for years, the best practice requires recognition of this force, since no foundation can be guaranteed impervious, nor masonry absolutely tight.

The usual method is to assume a value for uplift. A common assumption is that two-third full head operates at the heel, and zero at the toe, the resultant pressure being applied at the third point. The total force $= \frac{1}{2} \times \frac{2H}{3} = \frac{H}{3}$. This assumption was made for Wachusett, Boonton, Cross River, Croton Falls, Hinckley, Ashokan, Kensico, Loch Raven (Baltimore) dams, and studies by North Jersey District Water Supply Commission. San Mateo[7a] and Marklissa[8] dams were designed for full head acting on whole joint. The uneconomy of present practice is pointed out by Jorgensen;[2a] the more material added to offset uplift, the greater is the area offered to upward pressure; he would secure additional stability by buttresses. On Gileppe, uplift was allowed for by considering the specific gravity of the masonry as 1.3 instead of 2.3. William Cain recommends that amount of uplift be increased in this order: for foundations of granite, sandstone, stratified rock with horizontal seams, earth, and gravel.[7b]

Minimizing of uplift[2b] can be effected by: (*a*) denser concrete near the water face; (*b*) gunite on the water face; (*c*) deep cut-off wall at the heel; (*d*) grouting foundations (see p. 126); (*e*) drainage system; (*f*) treating reservoir bottom immediately upstream; (*g*) bank of impervious earth against upstream face for part of height. Several of these methods were utilized at Elephant Butte and Arrowrock dams by U. S. Reclamation Service, and no allowance for uplift was made. Additional means of reducing uplift on low overflow dams are a rear apron and cut-off walls (see p. 153).

Upstream Face Made Denser. At Elephant Butte[9] dam a 1-in. coat of mortar (1:2 mix), in four layers of ¼-in. each,[9] was applied by cement gun on a raft as the water rose. Gunite has been applied to multiple-arch dams also. On Don Pedro dam, below elevation −168 (Top. = elevation 0) hearting was of 1 part cement, 2½ parts sand, 6 parts gravel (¼ to 2½ in.) and 3 parts cobbles (2½ to 14 in.). The outer 3 ft. on the water face and outer 5 ft. on downstream face were of 1:2:6§ mix, poured without separate forms ahead of the hearting, so that there was blending in the contact.[10]

Drainage wells, provided in many dams since 1900, not only reduce uplift, but also intercept percolation which might otherwise disfigure the down-

* For reasons, see first edition of Waterworks Handbook, p. 119.
† See also Temperature effects, p. 122.
‡ Uplift was neglected in the recent La Loutre dam in the Canadian wilds; boring exploration indicated solid rock.[6] See also Table 50.
§ Coarse aggregate was made up of 4 parts gravel (¼ to 2½ in.) and 2 parts cobbles (2½ to 14 in.).

stream face. Drainage wells are less effective in winter, and below the ground-water level of the downstream fill. J. W. Ledoux[7c] prefers drainage provisions to use of excess masonry to resist assumed upward pressures. Drainage channels may defeat their purpose by facilitating percolation, or becoming clogged.

Effect on Designs. Eight cases may be assumed: (*a*) no upward pressure, *i.e.*, masonry and underlying rock impermeable; (*b*) upward pressure = h at heel, zero at toe, and varying uniformly between, uniformly distributed, and exerted upon one-third area of joint; (*c*) same as (*b*) but exerted upon two-thirds area of joint; (*d*) same as (*b*) but exerted upon full area of joint; (*e*) uniform upward pressure = h exerted upon full area of joint; (*f*) upward pressure = h at heel, h' at toe (backwater, or assumed ground-water level is h' above joint), varies uniformly between, is uniformly distributed, and exerted upon one-third area of joint; (*g*) same as (*f*) but exerted upon two-thirds area of joint; (*j*) same as (*f*) but exerted upon full area of joint. Computations in Table 42, shown in columns *a* to *j*, correspond to cases *a* to *j*, respectively. When u is less than $l \div 3$, upstream part of the joint for length $= l - 3u$, being in tension, according to the trapezoidal law, is supposed to be open, and so to be subjected to full reservoir pressure for the upper half of Table 42, but, in the lower half, the joint is supposed not to open under tension. The latter assumption is made to avoid complication introduced by considering $l - 3u$ to be under full head, so that the effect of a given upward water pressure exerted upon increasing proportions of area of joint may be more clearly seen.

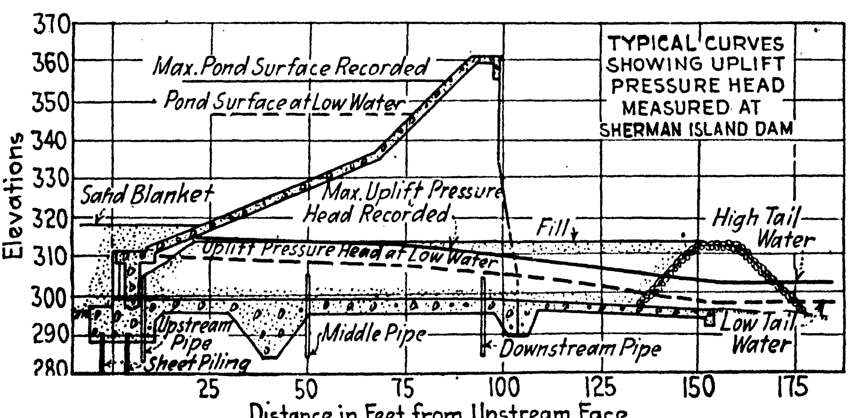


FIG. 34.[63]

Tests of uplift made in Germany[11] indicate distribution of pressure usually assumed is not true in that pressures at the toe may be as high as 50 per cent. of the head. Duryea[36c] measured uplift at Boquilla dam, Mexico, on a grouted limestone foundation as ½-head at heel and zero at toe. Test pipes for observation have been installed in recent dams, notably Brule River, Wis.,[12] and Oester dam, Westphalia.[11]*

Upward Pressures Measured on Brule Dam. This dam[12] is about 70 ft. high above the bottom of the cut-off trench and will impound 60 ft. of water. Underdrainage consists of 12-in. longitudinal drain, with 12-in. lateral drains at right angles, 20 ft. on centers. The foundation rock was a schist inter-

* See also Colman, *T. A. S. C. E.*, Vol. 80, 1916, p. 421.

sected by many minute cracks. No grouting was done. About half of the length of the dam is gravity section and the remainder power-house section. A row of 1-in. pipes to record height of water was installed in the gravity section at distances from back of the dam of 1 ft. 6 in., 7 ft. 6 in., and 19 ft. 6 in., respectively. The latter two were dry whereas the first one recorded a head averaging 60 per cent. of the reservoir head. In the power-house section, four pipes were installed between the scroll cases. One was 18 in. from the water face, a second about 18 ft. downstream, and two others in the piers supporting the superstructure. The two last recorded no head, while the first recorded 47.5 per cent. of the reservoir head, and the second, 30 per cent. (Information from D. W. Mead, Consulting Engineer.)

Table 42. Effect of Upward Water Pressure, As Shown by Change in Position of Resultant Reaction, and Variation in Its Intensity at Toe of Joint

Case			*a*		*b*		*c*		*d*	
Dist., ft. †	*l*	*l*/3	*u*	*r*	*u*	*r*	*u*	*r*	*u*	*r*
50	36.0	12.0	12.2	5.3	*9.9	*5.8	*8.7	*6.0	*7.9	*6.2
110	78.4	26.1	33.3	7.4	30.9	7.4	27.9	*7.4	*22.3	*7.7
170	132.5	44.2	56.1	11.8	......	......	......	......	......	......
200	149.7	49.9	59.1	16.4	......	......	......	......	......	......
Same, assuming joint does not open with tension at heel when $u<l/3$										
50	36.0	12.0	12.2	5.3	*11.3	*5.4	*10.2	*5.6	*9.0	*5.9
110	78.4	26.1	33.3	7.4	30.9	7.4	27.9	7.4	*24.1	*7.6
170	132.5	44.2	56.1	11.8	......	......	......	......	......	......
200	149.7	49.9	59.1	16.4	......	......	......	......	......	......

Case			*f*		*g*		*j*		*e*	
Dist., ft. †	*l*	*l*/3	*u*	*r*	*u*	*r*	*u*	*r*	*u*	*r*
50	36.0	12.0	{......	....	....	....	....	....	...... 7.9	+1.1 5.2
110	78.4	26.1	{......	....	....	....	....	....	...... 22.3	+3.0 5.5
170	132.5	44.2	51.7	11.6	46.6	11.3	*35.7	*11.7	...... 35.7	+4.8 8.7
200	149.7	49.9	55.1	15.7	*47.3	*15.4	*35.7	*16.2	...... 35.7	+5.8 13.0
Same, assuming joint does not open with tension at heel when $u<l/3$										
50	36.0	12.0	......	......	......	......	......	......	7.9	6.3
110	78.4	26.1	......	......	......	......	......	......	22.3	8.5
170	132.5	44.2	51.7	+0.4 11.6	...... 46.7	+0.7 11.3	*39.1	+1.1 11.4	...... 35.7	...... 13.5
200	149.7	49.9	55.1	+0.7 15.7	*49.1	+1.4 15.1	*40.2	+2.0 15.6	...... 35.7	...... 18.8

* Only quantities marked * are changed if joint does not open. Upper figures in "*r*" column under "*e*," "*f*," "*g*," "*j*," show correction for neglecting proportion of pressure at toe directly counteracted by back-water pressure. A *trial* profile for Wachusett dam (see p. 186) was used for above computation. Ice thrust (horizonal) = 47,000 lb. Backwater level, from which *h'* is measured. 138 ft. below top. Water level and ice 15 ft. below top. List of symbols, p. 130.

† From top to joint.

Earth pressures on the heel only are considered in designing. These may result from backfilling, grading, or silting of the reservoir (Chap. 6) and should not be neglected (see Fig. 35 for an illustrative diagram of pressures). This detail should receive consideration if any relatively large depth of earth is to be refilled or embanked against the upstream face of a masonry dam, as is not uncommon, particularly with non-overflow dams. If earth be selected and thoroughly compacted in thin layers, tight against the dam, the pressure may be less than the fluid pressure of saturated earth. Cain figures pressure from submerged fills[7b] as that of liquid mud, exerting the pressure of a fluid weighing 62.5 lb. per cu. ft.

Earth filling on toe may be saturated by high ground water and will cause appreciable loads in deep foundations. An advantage of an earth fill over the toe is the diminution of the effects of temperature. As the lower part of a dam carries the higher unit compressive stresses, the earth covering may be a distinct safety factor in high dams. Some engineers have advocated covering a large portion of the downstream face of a dam in this way, where conditions are suitable. Incidentally, the covered portions of the masonry, of whatever kind, need no finish, and so a small saving can be effected.

Temperature Effects and Cracks. Few masonry dams have no cracks apparent on the downstream face. Some close up after the first winter, probably due to the efflorescence of magnesia or lime from the cement. A leak on Crystal Springs dam became a mere "weep" overgrown with algae.[7d] The greatest difficulty in the construction of dams is to make the structure watertight (Jorgensen).[2c]

Setting concrete rises in temperature due to chemical reactions and, if placed in summer, may reach 100°F. (Paul and Mayhew.)[14a] The maximum is reached in 30 days and maintained for several months.[14b] Hardening and heating being contemporaneous, "the mass assumes its final form when at the highest temperature. Consequently there is no compressive stress in the masonry to be relieved by the subsequent cooling, the entire range in temperature between 100°F. and the lowest temperature to which the masonry cools, being converted into tensile stress" (A. J. Wiley).[14b]

Cracks result in an unreinforced concrete mass as soon as cold weather ensues. Although such cracks have never been known to cause the failure of a dam built of good concrete or other masonry, they undoubtedly weaken a dam in three respects: (*a*) water seeping through exerts uplift pressures which lessen the stability; (*b*) stabilizing value of the outer masonry is lessened by the disintegration promoted by the cracks; (*c*) there is probably a loss of valuable cement ingredients through leaching. Cracks in a dam on St. Croix River, Woodland, Me., were repaired by grouting and applying a Gun-crete facing.*

Temperature changes have been observed in many masonry masses by thermophone readings. The effects of temperature on the compressive stresses in a gravity dam are not established.[14c] It is proved analytically that a temperature rise in an arch dam produces shortening of the arch rib, which increases the compressive stresses. Temperature and setting effects must be complex in large dams, the various parts being placed under different

* Cement-Gun-Construction Co., Chicago.

temperature conditions. For thermophones in Kensico dam, see E. N., Dec. 10, 1914, p. 1172. For description of electric thermometers used in Arrowrock dam, see T. A. S. C. E., Vol. 79, 1915, p. 1227. Thermocouple readings on the thin arch dam at Lake Spaulding are recorded in *E. N. R.*, Aug. 9, 1917, p. 253. The Bureau of Standards is developing apparatus for recording temperatures and stresses. Methods employed by the arch dam committee of Engineering Foundation are given in *E. N. R.*, Dec. 11, 1924, p. 964.

Temperature Cracks. *Cross River Dam.* One and one-fourth-inch square steel bars were placed in horizontal layers in the upper 30 ft. Commencing at the top, these layers included six rows of rods in each of the first two courses, five in the third and fourth, four in the fifth, and two in the sixth. Nevertheless, temperature cracks appeared, three extending 70 ft. from the top, and a fourth, 43 ft. The cracks were widest at the top; total opening of all cracks was ⅞ in. Greatest leakage was 23.9 gal. per min. Cracks were calked on the upstream face with lead wool and grouted. Leakage was cut down to 2 gal. per min. but was never entirely stopped (1925). No horizontal cracking was indicated. *Croton Falls dam* was reinforced (see p. 179), but had three major and several minor cracks after the second winter following completion. Both dams continued (1925) to seep enough water to disfigure, generally, the downstream faces. Up to Mar. 17, 1906, five cracks of measurable size had appeared in *New Croton dam*, 1168 ft. long (finished in 1905); in the spillway section, 1000 ft. long, 10 small cracks. The average distance between main cracks is 200 ft.

Boonton Dam. In the Boonton dam, 2150 ft. long (completed 1903), there appeared, previous to Feb. 27, 1907, 17 cracks of considerable size and the same number of smaller cracks; the probable sum of their widths being not far from 3.5 in. Considerable seepage occurs at Boonton dam, keeping the face wet in summer and ice-covered in winter. Results of thermophone observations in the Boonton dam seem to warrant the following conclusions: In dams of this size, 12 months will elapse between the placing of the masonry and the assumption by the masonry of a temperature dependent on local exterior conditions; in other words, the heat generated by the setting cement only becomes negligible after 1 year. After the first year, the temperature of the interior will vary 30 days behind the atmospheric temperature. For distances up to 20 ft. from the face, the range of interior temperature varies nearly inversely with the cube root of the distance from the face. The temperature of a mass of masonry due to chemical action is highest 18 hr. after placing, averaging about 100°F. More cracks appeared in masonry laid in summer months. Approximate computations indicate that the ultimate tensile strength of the mass of a Portland cement dam is not far from 800 lb. per sq. in. (See Fig. 69, p. 178.)

Contraction joints have been provided in many recent dams (notably Ashokan, Kensico, East Park, Arrowrock, Elephant Butte), to localize vertical cracks, reduce secondary stresses, and improve appearance. They are spaced 50 to 100 ft. and provided with a keyed or metal water-stop. If held securely by a tongue and groove, longitudinal cracks may occur between the joints of a dam built in alternate sections, since the masonry in the section last placed, tending to shrink, will be constrained by the joint.

Temperature and Disintegration Allowance. A convenient method of taking account of temperature effects, possible disintegration due to freezing of water in joints or in pores of stone and mortar at the face of the dam, and the character of facing and mode of applying, is to assume that a layer of masonry exposed to the atmosphere cannot be considered as effective for resisting stresses other than those due to these causes. For conditions similar to northeastern United States, a thickness of 4 ft., normal to the face, would comprehend the greatest range in temperature, viz., 12° either way from the mean at the inner side of the layer and $32\frac{1}{2}$° either way from the mean at the exposed surface; the masonry back of the 4-ft. line would be subject to much smaller variations, which would produce stresses not excessive in good masonry. Extreme diurnal variations would be much greater than the averages named, but they would penetrate to slight depths. Assuming a variation of 12° from the mean at the 4-ft. line, a coefficient of linear dilatation* of 0.000,004 per °F., modulus of elasticity of 360,000 tons per sq. ft., and adjustment to a state of no internal stress at the mean temperature, there would be stresses of 17 tons per sq. ft., either tension or compression. But some experiments show much lower and others much higher moduli, so that the stresses may be estimated as low as 10 or as high as 30 tons for a variation of 12°. Movements caused by these stresses would be very small (for a mean value, about $\frac{1}{200}$ in. in 100 ft. for each ton per sq. ft.); and if the effect be distributed over a number of joints, as seems most probable, the opening, in case of tension, at any one joint would be minute. But since temperature stresses tend in part to relieve those caused by external forces, it is sufficient to assume a line 3 ft. back from, and parallel to, the portion of the face exposed to atmosphere. Although this layer is not included in the section for resisting external forces, its weight is borne by the "effective" masonry, for the face layer is not detached nor self-supporting. So small a part of the upstream face will usually be exposed to the atmosphere, and when so exposed will be subject to no stresses other than those due to the weight of the masonry and to temperature, that it is unnecessary to make a similar allowance here. Having thus provided for temperature, it is reasonable to allow higher unit stresses in the "effective section;" for in methods which make no separate allowance for temperature these stresses, along with indeterminate elements, are provided for by adopting low unit working stresses.

Table 43. Stresses Normal to Direction of Resultant, at Point Near Toe of Joint, Making Allowance for Temperature and Frost, by Assuming "Ineffective" Layer, Normal to Face of Dam

Joint	3-ft. allowance	4-ft. allowance
30 ft. below top	31.4 tons sq. ft.	†................
50 ft. below top	13.0 tons sq. ft.	20.7 tons sq. ft.
70 ft. below top	10.0 tons sq. ft.	12.2 tons sq. ft.
90 ft. below top	8.0 tons sq. ft.	10.0 tons sq. ft.
110 ft. below top	7.7 tons sq. ft.	9.1 tons sq. ft.

Assumed external forces are water and ice at 15 ft. below top and two-thirds upward water pressure; cross-section used for computation is that of Wachusett dam (see p. 186).

* Coefficient of linear dilatation is expansion per unit along any line, or direction.

† Resultant falls beyond limit of joint as reduced, *i.e.*, within 4-ft. "ineffective" layer.

Maximum pressures at the faces, computed in accordance with the trapezoidal law (p. 134), are probably in excess of the actual, except as greater pressures may be produced by constructing the faces of less compressible masonry than the interior, or by the expansion of the masonry in warm weather, or possible when wet and frozen.

FOUNDATIONS

Pressures. Foundations for solid gravity dams should be on rock, the bearing power and impermeability of which are known from investigations. Good foundation is one of the most important considerations for a masonry dam, as to both stability and cost. Only relatively low and unimportant dams may, under stress of circumstances, be placed on other than rock foundation, and such other foundation must be most intelligently selected and prepared. Foundation pressures are commonly restricted to 15 tons per sq. ft. on strong rock (cf. Tables 49 and 50), and to 10 tons on shale. Eel River dam, Cal.[15] is founded on shale which was grouted to a depth of 50 ft. below the base.

Failures. Among dams which failed by reason of foundation defects are Austin, Tex.;* U. S. Dam No. 26, Ohio River; Stony River dam, W. Va.;[69] reservoir wall, Nashville, Tenn.; and dam at Austin, Pa.[70] This last did not go down into rock, but only rested on or near surface. The rock layers were nearly horizontal, from 2 to 6 in. or more in thickness, and parted by unctuous clay. The coefficient of friction between two surfaces of rock separated by clay or soft, wet shale may not be more than from 0.3 to 0.5 while the dam would probably have failed if the coefficient had been 0.55.[7e] Foundations of Elwha dam, Port Angeles, Wash., blew out, leaving dam arching the opening.[16]

Earthquakes.† The earthquake of Apr. 18, 1906, cracked and shattered the calcareous sandstone under San Mateo dam and gave rise to numerous new springs in creek bed below; but with lapse of time these choked up and after a few years discharged an insignificant quantity. Dam with 30 thousand million gal. of water behind it, stood earthquake shock perfectly.[7d]

Investigations (see footnote on p. 118). Foundations should be thoroughly examined preliminarily by borings, test-pits, shafts, and drifts. The rock at Iron Canyon was examined by the Engineers of U. S. Reclamation Service by means of a leverage loading machine in a 3 by 6 ft. drift, 35 ft. into the cliff.[17] Foundations of Keokuk dam across Mississippi River were tested for porosity by using compressed air exerting four times the pressure of the anticipated head. Test holes, 4 in. in diam., and 30 ft. deep, were put down 36 ft. apart on the axis of the dam.‡

Excavation should be to a depth sufficient to uncover sound and tight rock, if practicable. Final excavating should be done without explosives, to reduce the chances of shattering the rock on which masonry is to be built. As foundations must penetrate alluvial deposits in the stream bed, trench sheeting or

* See p. 168.
† See also p. 826.
‡ QUERY. May not some air be objectionably pocketed in the rock so as to interfere with subsequent grouting, without being discovered? Instead why not pump water, noting closely the quantity and pressure, and changes in both?

bracing, stream control (see p. 157) and water handling are important. Adequate pumping equipment is essential. Wagoner[2d] reports that part of the work on the wrecked Walnut Grove dam was done in 3 or 4 ft. of water, in the absence of adequate unwatering equipment, and that the dam was founded, in part, on a boulder, mistaken for bed rock. For timbering designs for wide trenches, see diagrams by F. R. Sweeny, *E. N. R.*, Apr. 10, 1919, p. 708; tables in Vol. II, p. 284, *et. seq.* "American Sewerage Practice," by Metcalf and Eddy; also "Deep Trenches for Reservoirs," by J. M. Greig, *Can. Eng.*, Aug. 19, 1915, p. 269. An impervious stratum upstream allowed the excavation for the O'Shaughnessy dam, Cal., in the open by the erection of a small diverting dam. Excavations 63 ft. below the track were made by machines at the Miami works.[19] For short bibliography, see p. 160.

Reducing Seepage.* Cut-off walls are important to prevent flow between a dam and its foundation, and to prevent sliding, with all types of dams. For solid gravity dams, a relatively narrow trench (5 to 20 ft.) is excavated to satisfactorily watertight materials or to such depth, not less than 10 ft., as to offer great resistance to passing of water. Grouting is frequently done below the bottom of the trench, see p. 127. Commonly the cut-off trench is filled with dense masonry, built against the sides of the excavation and carefully bonded to the dam. Cut-off trenches are generally placed under the heel, but may follow an irregular line to meet local conditions. To reduce seepage around the dam, walls are sometimes extended beyond its ends. A wing wall was employed to prevent seepage through the abutting hills at Derwent reservoir, England, where the site of the reservoir was underlaid by strata of sandstone and shale, some of which were broken. This condition extended far into the hills. The cut-off trench was extended ½ mile along each side of the reservoir.[20] The foundations of Loch Raven dam are underdrained by a system of 6 in. pipes, 30 ft. center to center.[21]

Earth Foundations. It is not customary to build high masonry dams on earth foundations, because the pressure at the toe is so great that the earth would not offer sufficient resistance, but at the end of a dam enveloped by earth cones, there will be no excessive pressure at the toe; moreover, the weight of the embankment over the earth would tend strongly to prevent the earth from squeezing out. Under such circumstances, it is proper to build a portion of a masonry dam on very compact earth (hardpan or its equivalent) under competent engineering supervision.

Grouting Foundations. *Cross River Dam.* Drilling and blasting were allowed until seemingly hard rock was reached, when barring and wedging was resorted to for removing loose portions. At times this would disclose rotten rock below, calling for more blasting. The final bottom was washed clean, and sounded with a heavy hammer in the presence of an inspector. Mortar was spread over this wet surface just before masonry was placed. Wells of rubble masonry were built around springs. Vents were left for grouting, all springs being free to rise to their hydraulic grade line. A Douglas No. 2 bilge pump was used for grouting.

New Croton Dam. Seams, erosions, and caves in rocks were thoroughly excavated, then cleaned with a stream of water under high pressure from a

* See also "Minimizing of Uplift," p. 119.

nozzle. Larger spaces generally were packed with rubble masonry, smaller spaces poured full of grout of Portland cement and fine sharp sand, 2 to 1, wherever possible. Where pouring was not possible, grout, mixed 1 to 1, was pumped. Connections to some seams were made by drill holes; a few caves were entered by small shafts and tunnels. Water encountered under pressure was forced into pipes sealed into the rock and carried up above the level to which water rose, before grouting. Some of these pipes were 10 in. diam., but most were 2 in. Preliminary to grouting, pipes, etc., were thoroughly flushed with clean water under high pressure. One spring yielded 1.5 mgd. For one spring 2-in. pipes were carried up 90 ft. and, it proving impracticable to grout through so great a depth of water, plastic blue clay, well kneaded with water, was rammed in with a 2000-lb. pile driver with maximum drop of 4 ft., to a total of 22 cu. yd. (For further details, see "Foundations of New Croton Dam," by C. S. Gowen, *T. A. S. C. E.*, Vol. 43, 1900, p. 469.)

The Clackamas River dam of the Portland, Ore., Ry., Light & Power Co., is founded on volcanic andesite and basalt, heavily fissured. A double line of holes were drilled under the heel of the dam to average depth of 50 ft. Canniff pneumatic grout mixer* and injector was used, with air pressure from 50 to 200 lb. per sq. in., depending on the tightness of the hole; high pressure caused blow-outs at surface. Holes were $2\frac{3}{8}$ to 8 in. diam.; 3-in. wrought-iron pipe casings, with threaded tops for attaching hose, were put down for upper 4 to 6 ft., and grouted in. Grout contained 2 to 15 parts water to 1 cement. A flexible copper hose connected the grout machine to the pipe. Introducing the hose into the pipe for grouting low parts was tried, but did not prove worth extra work. Holes were tested under a head exceeding the normal reservoir head; in some cases, communication was found between holes 70 ft. apart. Drilling and grouting 555 holes, aggregating 34,038 ft., cost $1.55 per ft.[67]

At Olive Bridge dam† of Catskill Water Supply for New York, 29 grout holes, 3 in. diam., 5 ft. apart, and 13 to 20.8 ft. deep, were drilled in the cut-off trench down to a seam indicated by previous borings. When this seam was tapped water rose under pressure to the floor of the cut-off trench. Two-inch grout pipes were chipped with a chisel at the lower end, wrapped with jute for about 8 in. and driven 3 ft. into the drill holes, the jute on the lower end fitting the hole tightly. The annular space outside of the pipe was grouted, and filled with more jute wound around the pipe and calked so firmly that all flow of water stopped. Pipes were finally sealed to rock with neat Portland cement. As the cut-off trench was filled, the 2-in. grout pipes were carried up with the concreting to a proper height for grouting. A Cockburn-Barrow grouting machine of about 4 cu. ft. capacity was used. This machine consists of a cylinder, 1.75 ft. diam. and 3 ft. long, fitted with paddles to churn the grout. Materials are introduced through a hole in the top. Pipes admit air under pressure, which forces grout through a 2-in. hose, connecting by a tee to the grout pipe. A throttle valve in the tee permits the blowing off of the hose without disconnecting it from the pipe. Air pipes are controlled by screw valves; outlets to hose, by throttle valve. A pressure gage is attached to the air pipe. A compressed air engine attached to the machine keeps the stirring

* Manufactured by Ransome Concrete Machinery Co.

† See also "Grouting Operations, Catskill Water Supply," by J. F. Sanborn and M. E. Zipser, *T. A. S. C. E.*, Vol. 83, 1919–20, p. 980. A hydraulic machine for grouting is described in *E. N. R.* July 14, 1921, p. 74.

paddles in motion. Prior to grouting, water stood 20 ft. below the tops of the pipes. Under 25 lb. pressure, water was immediately forced from two adjacent holes, spurting 6 ft. above the pipe mouths. Air pressure was then connected to the grouting machine and the holes grouted. The volume of pipe and drill hole was about 1.4 cu. ft.; 3.2 cu. ft. of grout were required to fill them.

At Elephant Butte dam, U. S. Reclamation Service grouted drill holes below cut-off wall and provided two rows of drainage wells downstream from this grout barrier, through which seepage could be detected. The sandstone bottom consisted of layers, badly fissured, separated by shale which disintegrated on exposure to air. Test holes were put down 20 ft. or more to assure satisfactory layers below; 4-in. openings, spaced 10 ft. center to center, were carried up in the masonry, and through them rock drills operated to drill holes into rock to a depth of 45 ft., for high-pressure grouting.[22]

DESIGN OF SOLID GRAVITY DAMS* (NON-OVERFLOW TYPE)

Judgment of Designer. Here can be given only working formulas with such explanation as may be needed for intelligent use by an engineer who has some knowledge of the subject. In assigning values to permissible pressure on masonry and to ice thrust, and in making various allowances not subject to exact calculation, *e.g.*, upward pressure, but which affect the thickness and height of the dam, the judgment of the engineer must be guided by the importance of the dam, quality of available materials and workmanship, local climate, nature of the site, consequences of partial or total failure, and funds at his disposal.

Failure and Safe Design. A gravity masonry dam may fail by: (*a*) overturning about edge of any joint, due to line of action of resultant passing beyond limits of stability; (*b*) crushing of masonry or foundation; (*c*) shearing or sliding on foundation or any joint, due to horizontal thrust exceeding shearing and frictional stability; (*d*) rupture of any joint due to tension. (See also p. 146.) Hence the following limitations are generally established: (1) Lines of pressure, for reservoir full or empty, must not pass outside middle third of any horizontal joint. (2) Maximum normal working pressure on any horizontal joint must never exceed certain prescribed limits, either in masonry or in foundation. (3) Coefficient of friction in any horizontal joint, or between dam and foundation, must be greater than tangent of angle which resultant makes with vertical; *i.e.*, coefficient should be greater than $\frac{1}{2}h^2 \div W$. Investigate each assumed joint for the worst conditions of loading, reservoir both full and empty.

Freeboard, the height of top above full-reservoir level, varies from 5 to 20 ft.; seldom less than 10 ft. Brodie[23] concludes that for a given top width the superelevation affects not the quantity of material but only its distribution in the cross-section.

Symbols for masonry dam formulas (see Fig. 36) grouped alphabetically. Unit of weights and forces is weight of 1 cu. ft. of water. All lengths are in feet. (These symbols apply only to pp. 130 to 135.)

* For detailed instructions, see Creager, "Engineering for Masonry Dams," John Wiley & Sons, Inc., 1917.

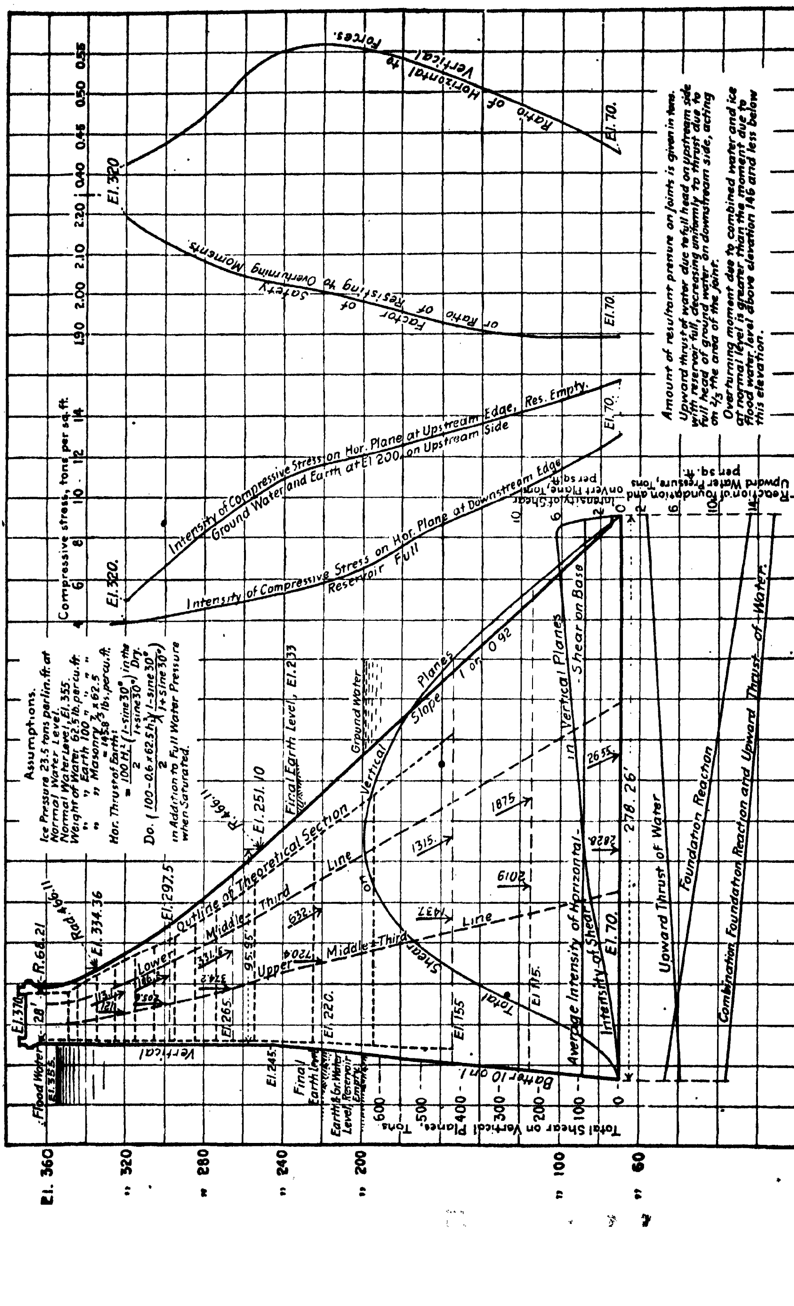


FIG. 35.—Kensico dam at maximum section. Investigation of stability under various conditions.

B = breadth, or thickness, top of dam.
b = horizontal distance between toe of l and toe of l_o (may = 0 for upper part of dam if downstream face is vertical for any distance below top).
C.G. = center of gravity.
c = coefficient of upward water pressure.
ch = intensity of pressure of water vertically upward at upstream side of joint; it is considered to decrease uniformly to zero at downstream side.
d = depth of water on top of dam.
D = horizontal dynamic thrust of water on back.
E = thrust of earth on downstream face.
f = distance from nearer face of joint, to point of application of resultant.

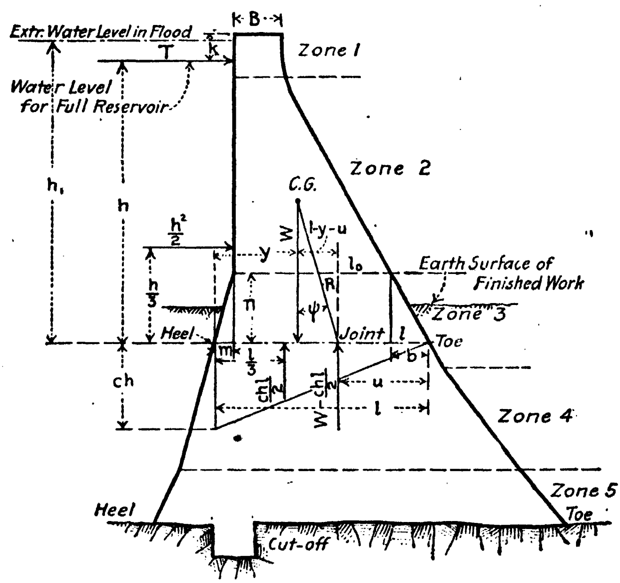


Fig. 36.—Typical section, non-overflow type.

h = head, or vertical depth, from water level for full reservoir to any joint, real or imaginary, under computation.
h_1 = head from extreme water level in flood, to joint, or to surface of mud.
h_2 = depth of mud against upstream face.
$h^2/2$ = horizontal thrust due to static pressure of water against the upstream face of the dam.
$h_1^2/2$ = ditto for water at flood level.
$H = h + k$ = vertical distance from top of dam to joint under computation.
h' = head due to backwater, downstream face.
H_1 = depth of fill, downstream side, above joint.
k = height of dam above full reservoir level.
l = length of joint under computation.
l_o = known length of joint next above, *i.e.*, the one just previously determined.

m = distance from heel to line of vertical upstream face (= zero for portion having vertical upstream face).
M = moment of horizontal force about any point in l.
n = vertical distance between joints l and l_o.
p = intensity of vertical pressure at toe.
P_r = maximum intensity of vertical pressure at face of joint, tons per sq. ft.*
q = intensity of vertical pressure at heel.
r = intensity of pressure at toe of joint, parallel to R.
R = resultant of forces on joint, reservoir full.
R' = resultant of forces on joint, reservoir empty.
s = intensity of shear on joint, tons per sq. ft.
T = thrust of ice at water surface, considered horizontal (never combined with h_1).
u = distance from toe to intersection of R with joint (when $u < l/2$ it has same value as f).
V = downward water pressure on upstream face.
V_1 = downward water pressure on downstream face.
v = distance between intersections of R and R' with l ($=l - y - u$).
w = weight of masonry above joint; for slice of dam 1 ft. thick, $w = \Delta$ times area of cross-section above joint.
w_0 = weight of masonry above l_o.
W = algebraic sum of all vertical forces acting on joint, omitting uplift.
y = distance from heel to intersection of W or R' with joint (when $y < l/2$, it has same value as f).
y_o = value of y for joint l_o.
z = distance from heel to line of action of V.
Toe = downstream edge of joint; Heel = upstream edge of joint.
α = angle of slope of downstream face from horizontal.
γ = unit weight of water = 62.5 lb. per cu. ft.
γ' = unit weight of mud = 75 to 90 lb. per cu. ft.
δ = angle of E from horizon.
Δ = unit weight of masonry, taken as 156.25 lb. per cu. ft. = $2\frac{1}{2}\ \gamma$ in many computations. Varies from 145.5 to 166.0.
ρ = specific gravity of masonry, taken as 2.5.
ϕ = angle of slope of downstream face from vertical.
ψ = angle between R and vertical.

Procedure. Horizontal plane joints are assumed at various convenient elevations, or distances below top, for purposes of computation. Moments of forces taken about intersection of line of action of W with joint, the forces being represented by their resultants and considered to act in the same vertical plane. A cross-sectional slice 1 ft. thick is used for computation and so the forces are those acting upon 1 lin. ft. of dam. Begin with assumed profiles and modify, if necessary. The following formulas determine the position, under various assumptions, of the intersection of the resultant of the forces acting on the dam with the joint in question. In other words they determine the position of the "line of pressure." (See pages 133 to 135 for suggestions.)

* In practice, this varies from 8 to 23 tons. Cf. tables, pages 161 to 167. A conservative value is 15.

General Equations for Stability against Overturning.*

$$Th + \frac{h^3}{6} + \frac{chly}{2} - \frac{chl^2}{2} - (l - y - u)\left(W - \frac{chl}{2}\right) = 0.$$

1st. Case.—Water pressure only, water at full reservoir level. $T = O$. $c = 0$

$$l - y - u = \frac{h^3}{6W} \text{ or } \frac{h^2}{6\frac{W}{h}}$$

2d Case.—Water pressure and ice pressure, water at full reservoir level, $c = 0$.

$$l - y - u = \frac{h}{6W}(6T + h^2) \text{ or } \frac{6T + h^2}{6\frac{W}{h}}$$

(For any dam, $6T$ is constant, after a value for T has been selected.)
3d Case.—Water pressure and upward pressure beneath joint. $T = 0$.

$$l - y - u = \frac{h^2 + cl\,(3y - l)}{6\frac{W}{h} - 3cl}$$

4th Case.—Water pressure, ice pressure, and upward pressure.

$$l - y - u = \frac{6T + h^2 + cl\,(3y - l)}{6\frac{W}{h} - 3cl}$$

5th Case.—Water pressure only, water at flood level. $T = 0$. $c = 0$.

$$l - y - u = \frac{h_1^3}{6W} \text{ or } \frac{h_1^2}{6\frac{W}{h_1}}$$

Another General Formula† is:

$$u = l - y - \left\{(h_1 + h_2)[3h_1h_2 + 6T - 3d^2 + cl(3y - l)] + h_1^3 + \frac{h_2^3\gamma'}{\gamma} + 2d^3 + 3D(h_1 + 2h_2 - d) - 6V(y - z) + 6E\left[(l - y)\sin\delta - \frac{H_1}{3}\frac{\sin(\delta + \alpha)}{\sin\alpha}\right]\right\} \div [6(V + E\sin\delta + w) - 3c(h_1 + h_2)l].$$

If any condition is to be omitted, its factor or term becomes 0 and the general formula must be simplified accordingly. *General Conditions:* (see Fig. 37).

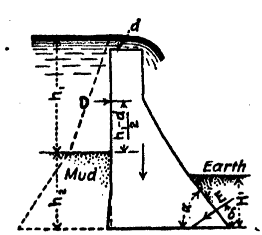


FIG. 37.

Horizontal static water pressure on back (head $= h_1$); upward water pressure on base, pressure intensity decreasing uniformly from $c(h_1 + h_2)\gamma$ at heel to zero at toe; mud (liquid) pressure on back (head h_2); dynamic pressure of water, $D\gamma$; water flowing over top of dam, weight of water, of depth d, on top of dam being neglected. For condition of water not overtopping dam, $d = 0$, and $D = 0$; no dynamic pressure, $D = 0$; no upward water pressure, $c = 0$; no mud (*i.e.*, mud replaced by water), $h_2 = 0$. If ice be disregarded, $T = 0$. If H_1 is of great depth and the downstream slope varies considerably, an approximate solution is possible by assuming an average slope beneath

* T and W in these equations are in terms of weight of 1 cu. ft. of water.
† All forces are in terms of weight of 1 cu. ft. of water.

the earth. The expression for earth thrust is general. After u is determined for each joint, p can be determined, the general expression corresponding to above expression for u being:

$$*p = \frac{2\gamma}{l}\left[V + E\sin\delta + W - \frac{c(h_1 + h_2)}{2}l\right]\left(2 - \frac{3u}{l}\right).$$

In computations for p it is necessary to obtain the position of the centroid of a trapezoid with respect to the heel of the joint. In Fig. 38,

$$x = \frac{(l^2 + ll_o + l_o^2) + m(l + 2l_o)}{3(l + l_o)}$$

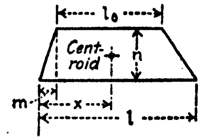


FIG. 38.

Shearing stresses† are emphasized in Atcherly's analysis,[71] which has not had wide acceptance. See Unwin in Eng., Vol. 79, 1905, p. 825.

Controlling Factors. For lower part of a high dam, when choosing p and q,‡ and slopes of faces, the bearing capacity of the rock foundation must be taken into account. It is convenient, especially for construction, to make upstream face vertical to as great depth as is permissible. Considering weight of masonry only and assuming uniform distribution of pressure over whole base or horizontal joint, the limit of height with vertical face might be limit of pressure in lb. per sq. ft. ÷ wt. of masonry per cu ft. Adopting 20 tons and 156.25 lb. respectively, height =256 ft. Other considerations greatly reduce this height. Flood-level conditions control beyond a certain height (usually greater than 100 ft.) depending upon excess of h_1 over h, and values assigned to T, B and k. In climates where T may be neglected h_1 is used in determining the section, for a dam stable with h_1, would be more stable for h. Having allowed for floods and waves, choice of k is arbitrary or limited by some local condition; B is fixed by necessities of footway or vehicles, usually; but k and B together are very directly affected by T.

Design of High Dams by Zones. Unwin pointed out the futility of attacking the problem by pure mathematics.[24] However, a section may be divided into zones, and mathematically investigated. The type of cross-section for usual conditions being now well known (see profiles on p. 168 etc.), the simplest procedure is to draw a trial section, test it by the formulas and modify until all conditions are satisfied. For a low dam (not a *very* low one) use upper part of cross-section for a high dam subject to similar conditions. Formulas and discussions given here apply to the now generally adopted type of retaining gravity masonry dams. With various modifications, the cross-section is a triangle, essentially, although the top always has substantial width§ and one or both faces may be curved or broken lines. With proper interpretation many of the formulas can be applied to dams of different shape. A very high masonry dam, for purposes of computations relating to design, may be divided into five horizontal zones, beginning at top, in each of which the limits of all conditions included in computations remain constant. The level at which one or more of these limits is reached fixes the bottom of each zone. Low

* All forces are in terms of weight of 1 cu. ft. of water.

† See also William Cain, "Stresses in Masonry Dams," *T. A. S. C. E.*, Vol. 64, 1909, p. 208, to which reader is referred for discussion of shears.

‡ For symbols see pp. 130 and 131.

§ See "The Economical Top Width of Non-overflow Dams" by W. P. Creager, T. A. S. C. E., Vol. 80, 1916, p. 723.

dams may have only the upper one or two zones. As will appear below, heights of zones depend upon allowed unit pressures in the masonry and upon other elements of design. Investigate sections in 10-ft. lifts for the upper 80 ft., and in 20-ft. lifts below. In preliminary studies, larger intervals may be employed.

Zone 1. A low dam or top portion of a high dam; rectangular cross-section (or substantially so); both faces vertical or with only *slight* batter. Limited by level at which downstream face begins to slope in order to keep point of application of resultant of forces on joint within middle third with reservoir full.

Zone 2. Bottom is level at which upstream face begins to slope to keep point of application of resultant of forces on joint within middle third with reservoir empty. Unless slope of upstream face exceeds 4 on 1, ignore the vertical pressure of water on it.

Zone 3. Bottom is level below which downstream face slopes more flatly than in zone 2, limit of pressure intensity on masonry at toe having been reached for this slope with reservoir full.

Zone 4. Bottom is reached when pressure intensity on masonry at heel attains limit with upstream slope being used; reservoir empty. Below this level upstream face must have flatter slope.

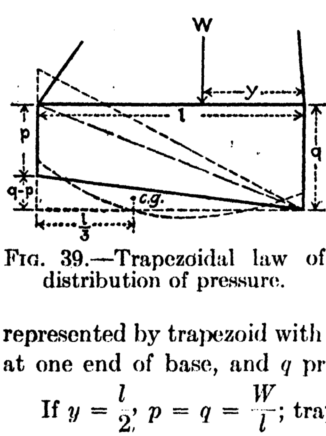


FIG. 39.—Trapezoidal law of distribution of pressure.

Zone 5. Slopes of both faces fixed by allowed pressure intensities on masonry at toe and heel, respectively; reservoir alternately considered full and empty. Limited only by necessities of dam site or strength of masonry. Only very high dams have fifth zone.

Trapezoidal Law of Distribution of Pressure. Assumption: incompressible wall on incompressible base. If W is vertical resultant of pressures, omitting uplift, its distribution is represented by trapezoid with p proportional in length to intensity of pressure at one end of base, and q proportional to intensity at other end.

If $y = \frac{l}{2}$, $p = q = \frac{W}{l}$; trapezoid becomes rectangle.

$y > \frac{l}{3}$, $q = \frac{2W}{l}\left(2 - \frac{3y}{l}\right)$; $p = \frac{2W}{l} - q$

$y = \frac{l}{3}$, $q = \frac{2W}{l}$; $p = 0$; trapezoid becomes triangle.

$y < \frac{l}{3}$, $q = \frac{2W}{3y}$; p becomes negative and there is tensive stress represented by the small triangle above the base, its maximum intensity being $-p = \frac{2W}{3y} - q$. Actually the distribution of pressure is probably represented more nearly by the figure with the curved bottom, and ends less than p and q, but we have not the knowledge necessary for constructing this figure.

If uplift is included, substitute u for y; for W substitute, $W - \frac{Chl}{2}$.*

* For a general formula, see p. 132.

If W is expressed in cu. ft. of water, W_w, the intensities of pressure, P_r, in tons per sq. ft. at nearest face, have values as follows:

(1) $f > \frac{l}{3}$, $P_r = \frac{W_w}{16l}\left(2 - \frac{3f}{l}\right)$

(1) $f = \frac{l}{3}$, $P_r = \frac{W_w}{16l}$

(3) $f < \frac{l}{3}$, $P_r = \frac{W_w}{48f}$.

CURVED OR ARCHED DAMS

Sites. Proper topographical and geological conditions are essential. The arch dam is particularly suitable to narrow canyons with approximately vertical bare rock sides. This type was rejected for Tallulah Falls dam because doubt existed as to reliability of the supporting strata in the abutments.[25] At Gibraltar dam, where no rock existed at the top to take the thrust of one end, an artificial abutment was furnished by pouring a 3000-cu. yd. block of concrete 1 year in advance of the dam.[26]

Advantages. No arch dam has been known to fail except because of faulty foundations. Effects of uplift may be disregarded. Brevity of construction period may simplify stream control; Corfino dam in Italy, 110 ft. high, containing 1,000,000 cu. yd. of masonry, was built in 65 days.[27] Reduced pressures on foundations suggest use, under proper restrictions, where a gravity dam could not be sustained. An arch dam was placed between two wings of gravity type on an Australian site where faulty rock in the stream bed was considered incapable of sustaining a heavy gravity section.[28] For dams of medium or great height, provided the length does not exceed 600 ft., and radius and central angle are so chosen as to give maximum probable arch resistance, the arch form offers economy. The saving in masonry is important; sparsely settled regions in the West could not have afforded costly, solid gravity dams. Increased safety resulting from an arched plan often has been utilized for gravity dams, notably Roosevelt and San Mateo (see p. 182). But curving in plan does not increase stability in all instances and may increase cost.*

Disadvantages. For very low dams, the arch form is generally not economical. On high dams, the slender cross-section is sometimes alarming to conservative minds. There is much difference of opinion as to proper formulas for designing; waste of materials is difficult to prove. Arch action in heavy sections is not so pronounced along the upstream face; hence axial compression is not operative to tighten the dam to the same extent as in this sections. This type can be used for an overfall dam only when the toe is protected against erosion. A 70-ft. dam at Crowley Creek,[29] Malheur County, Ore., is protected by a pool 35 ft. wide formed by a subsidiary arch 15 ft. high. See also p. 148.

Cracks in Arch Dams. No arch stress can be transmitted across open vertical cracks. Hellstrom† and others call attention to serious cracks in many existing arch dams; arch action has been destroyed. Stability in such

* See E. N. R., June 12, 1924, p. 1020; and July 31, 1924, p. 193.
† See paper on "Swedish Arch Dams," World Power Conference, July, 1924.

condition is dependent on cantilever action, until the rising water deflects the vertical cantilevers so as to close the cracks.

Formulas for Design. No generally accepted method of computation exists. A common assumption* considers an arch dam composed of both horizontal arches, and of vertical cantilevers fixed at the foundations. Division of load between arches and cantilevers introduces complex secondary stresses. A committee of Engineering Foundation is making observations on existing dams, and has built a test dam.[29a] Until this committee reports† (or other experimental facts are gotten), it is inadvisable to abandon old formulas. Values of moduli of elasticity, ranges of temperature in the masonry, distribution of the water load between arch and cantilever action, secondary arch action, shrinkage of concrete, and fixity of the arches, all are assumed with small basis of ascertained fact.[29b] W. H. R. Nimmo cites 16 considerations generally neglected.[29c] Hawgood does not underestimate mathematical analysis, but warns that results cannot be more accurate than the assumptions.[2h]

Cylinder formula has been used for many existing dams, none of which has failed.[29d] $T = \frac{RP}{S}$; T = arch thickness, ft.; R = radius, upstream face, ft.; P = water pressure, tons per sq. ft.; S = allowed stress on masonry, tons per sq. ft. William Cain discusses this formula in *T. A. S. C. E.* Vol. 85, 1922, p. 273. G. S. Williams [29e] characterizes it as "not even approximately correct for thin dams."

Arch and Cantilever Method (Edwin Duryea, Jr.).[30b] In a masonry dam, any point, A, is deflected downstream as the water rises. In an arch-type dam displacement of A corresponds to the deflection of a cantilever beam of unit length between two transverse vertical sections, and to the deflection of an arch lamina of unit height between two horizontal planes. The total water pressure will be shared between arch and cantilever, directly as respective rigidities or inversely as deflections. Cantilever beams are assumed fixed in direction at base, with planes before flexure still planes after flexure; arch laminae are assumed to be two-hinged, or changeable in direction at each bank. These assumptions are contradictory, and both tend to reduce the apparent proportion of pressure carried by arch action.

1. Vertical section through A:

   h = height of dam from base (below A) to crest,
   b = thickness of dam at base (below A),
   x = height from base of dam to any horizontal section,
   l = thickness of dam at height, x. $[= \frac{1}{2}(h - x)]$

2. Horizontal section through A:

   s = span of horizontal arch lamina before being shortened by pressure,
   r = mean radius of horizontal arch lamina before being shortened by pressure $= R - \frac{l}{2}$,
   l_1 = chord of half of horizontal arch lamina before pressure is applied,
   l_2 = chord of half of horizontal arch lamina after pressure is applied,

* See Duryea below.
† Progress bulletins have been published by Engineering Foundation.

T = thrust on horizontal arch lamina due to proportion of water pressure borne by it,

e = total shortening of chord l_1 due to thrust T; y_1 and y_2 = ordinates corresponding to l_1 and l_2,

$D = y_1 - y_2$ = deflection of point A.

3. Water pressures:

γ = weight of a cubic foot of water = 62.5 lb.,

γ_g = proportion of unit weight of water (multiple of 62.5) borne by gravity action,

γ_a = proportion of unit weight of water (multiple of 62.5) borne by arch action.

4. Compression of masonry:

E = modulus of elasticity = 1,500,000 lb. per sq. in.

$$\frac{h^3\gamma_g}{b^3E}x^2 = r - \sqrt{r^2-\frac{s^2}{4}} - \sqrt{\left[\frac{s^2}{4}+\left(r-\sqrt{r^2-\frac{s^2}{4}}\right)^2\right]\times\left[1-\frac{\gamma_a(h-x)}{lE}r\right]^2-\frac{s^2}{4}}.$$

After selection of any particular arch lamina, the only unknown is γ_a. Seven-place logarithmic tables are not sufficiently accurate; compute by ordinary multiplication.

A formula for arch deflection by Vischer and Wagoner,[31] is much simpler:

$$e = DV$$

e = total shortening under arch pressure of half the arch lamina;

D = deflection of center point of arch lamina;

α = one-quarter of total angle (in radians) subtended by arch lamina.

From theory of compression of elastic bodies,

$$e = \frac{T}{lE}l_l, \qquad D = \frac{\gamma_a(h-x)r}{lE\alpha}l_l,$$

where the vertical thickness of the arch lamina is unity, and l_l represents the half length of the arch lamina instead of the length of the half chord.

For a triangular profile with base = ½ height, and water level with apex:

$$\gamma_a = \frac{125x^2}{r^2 + 2x^2}.$$

Constant-angle Arch Dam* is suited to canyons in solid rock narrower at bottom than at top. A constant-angle dam acts more nearly like an arch, can deflect more without producing vertical tension in the upstream face, and, under equal conditions, is a safer structure than a constant radius structure. The main claim for economy is keeping the subtended angle of the arch as nearly constant as possible so that arch action is utilized to the maximum at each elevation.[32] This necessitates abandonment of the constant upstream radius generally employed, and the substitu-

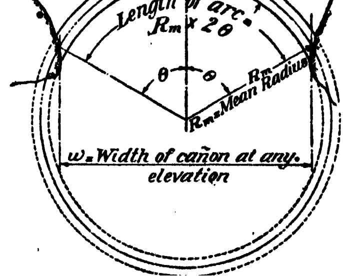


Fig. 40.

* Patents controlled by Constant Angle Arch Dam Co., San Francisco.

tion of radii of various lengths for successive zones determined by the width of the canyon at each zone. There are practical limits to the rigidity of this requirement. Jorgensen[2e] says it is not practicable near the base; Parker says that sites are rare where local conditions will allow the use of a constant angle; J. S. Eastwood[2f] believes application is extremely limited.

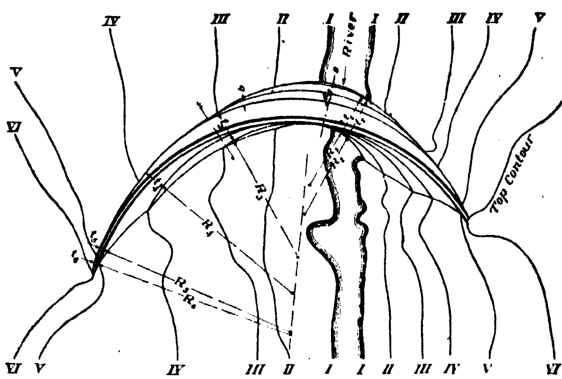


Fig. 41.—Paper location of constant-angle arch dam.

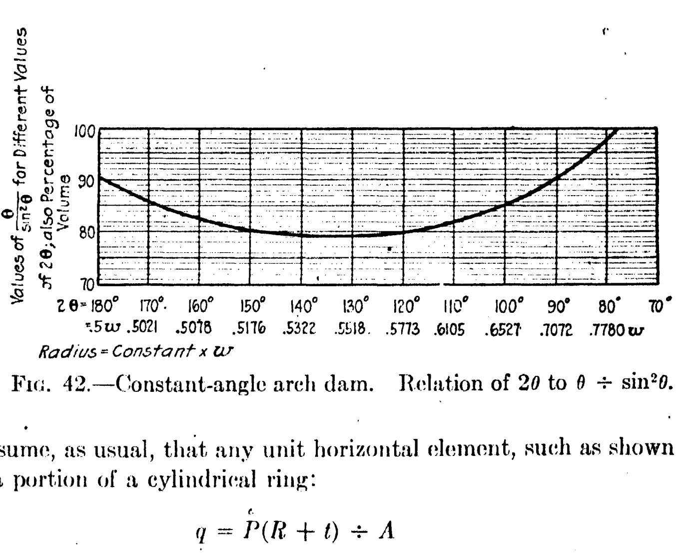


Fig. 42.—Constant-angle arch dam. Relation of 2θ to $\theta \div \sin^2\theta$.

Assume, as usual, that any unit horizontal element, such as shown in Fig. 40 is a portion of a cylindrical ring:

$$q = P(R + t) \div A \tag{1}$$

t = thickness of the dam; R = radius of downstream face; A = area of section at any given point; q = av. stress, lb. per sq. ft.; P = radial load, lbs. per sq. ft.

The volume of masonry is equal to the area of the section, times the mean radius, times the enclosed angle.

$$V = A \times R_m \times 2\theta \tag{2}$$

$$= \frac{C' \times \left(\frac{\omega}{2}\right)^2 \times 2\theta}{\sin^2\theta} = \frac{K\theta}{\sin^2\theta} \tag{3}$$

in which C' and K are constants, the latter depending upon the width of the canyon.

Table 44. Details of Some of the Principal Curved Masonry Dams*

Arranged in order of height. See sections on pp. 140 and 187.
Dimensions of following curved dams will be found on pp. 161 to 167; Arrowrock, Barren Jack, Beetaloo, Burraga, Butte, Chartrain, Chemnitz, Furens, Hemet, Gileppe, Granite Springs, Hijar, Roosevelt, San Mateo, Thirlmere, Turlock, Upper Otay, Urft, Villar, Wigwam, Zola. Section of Halligan dam, p. 187.

Place	Max. height above foundation, ft.	Base thickness, ft.	Top thickness: Thickness, ft.	Top thickness: Depth below crest where measured, ft.	Surcharge allowed ft. of water	Max. arch stress, lb. per sq. in.‡	Radius, upstream face, ft.	Length, ft.	Rock in foundation	Date
(a) Katoomba, N.S.W.....	25.0	20.29	3.0	0	1.0	233	220.0	320	Sandstone.........	1905
(a) Picton, N.S.W.........	28.0	13.62	7.0	0	10.0	186	120.0	112	Sandstone.........	1897
(p) Sorell Cr., Tasmania...	30.0	6.0	2.0	0	— —	208	110.0	155	— — — — — — — — — —	1912
(c) Winchester, Ky........	31.0	8.58	4.83	0	— —	498	318.4	407	— — — — — — — — — —	— —
(a) Queen Charlotte Vale, N.S.W.	32.0	8.65	3.0	0	— —	155	90.0	113	Quartzite..........	1898
(a) Parkes, N.S.W........	33.5	13.5	3.0	6.0	5.0	373	300.0	540	Granite...........	1897
(a) Lithgow, No. 1, N.S.W.	35.0	10.88	3.5	3.5	3.5	155	100.0	178	Sandstone.........	1896
(a) Wollongong, N.S.W....	42.0	11.62	3.5	5.0	1.0	311	200.0	535	Basalt.	1898
(a) Cootamunda, N.S.W...	46.0	13.0	3.0	8.0	1.0	389	250.0	640	Granite...........	
(a) Wellington, N.S.W.....	48.0	10.0	3.0	7.0	2.0	311	150.0	....	Conglomerate......	1899
(a) Mudgee, N.S.W.......	50.0	18.0	3.0	5.0	1.0	311	253.0	....	Altered slate.......	1899
Las Vegas, N.M.......	50.0	15.5	5.0	0	— —	350	250.0	210	Sandstone.........	1910
(a) Parramatta, N.S.W....	52.0	15.0	4.8	0	2.0	223	160.0	225	Sandstone.	— — —
(a) Lewiston, Idaho.......	55.5	14.5	5.33	0	— —	475	286.5	288	— — — — — — — — — —	— —
(a) Tamworth, N.S.W.....	61.0	21.5	3.0	3.0	2.0	311	250.0	440	Granite...........	1898
(b) Bear Valley, Cal.†.....	64.0	8.4(h)	2.75	0	— —	825	335.0	300	Granite...........	1884
(a) Medlow, N.S.W.......	65.0	8.96	3.5	21.0	3.0	186	60.0	124	Sandstone.........	1906
(o) Crowley Cr, Ore......	70.0	5.16	3.0	0	2.0	— —	70.0	158	Rock..........	1911
(a) Lithgow, No. 2, N.S.W.	87.0	24.0	3.0	3.0	3.0	155	100.0	221	Sandstone.........	1906
(d) Ithaca, N. Y.......(f)	90.0	7.75	— — —	0	— —	283	67.85	— —	Shale.............	1903
(b) Sweetwater, Cal.......	94.0	46.0	12.0	0	— —	188	222.0	380	Porphyry	1888
Las Vegas (proposed)...	95.0	43.3	5.0	0	— —	300	250.0	390	— — — — — — — — — —	— —
(n) Columbus, O. (Scioto R.)	95(m)	64.3	11.0	0	0	— —	800.0	1806	Limestone	1905
(b) Barossa, S. Australia...	113.0	34.0	4.5	0	— —	242	200.0	470	Shale	1903
(b) Zola, France..........	123.2	41.82	19.02	0	— —	196	157.0	205	— — — — — — — — — —	1843
(r) Salmon Cr., Alaska....	168.0	47.0	6.0	0	— —	330	331	644	— — — — — — — — — —	1914
(q) East Canyon Cr., Utah.	190.0	26.25	5.0	0	— —	174	98.8	144	Limestone.........	1915
(s) Hartebeesfoort, S. Afr.	194	73.0	15	0	— —	180	148–240	— —	— — — — — — — — — —	1923
(d) Lake Cheesman, Col...	218.5	176.0	18.0	0	0	200	400.0	710	Granite...........	1900
(e) Pathfinder, Wyo.......	210.0	72.66(k)	10.0	0	— —	181	150.0	425	Red granite	1909
Shoshone, Wyo.	328.0	108.0	10.0	0	0	— —	150(l)	175	Granite	1910

Principally from *E. R.*, Oct. 8, 1910, p. 403. (a) *E. N.*, May 19, 1910; also *Proc. Inst. C. E.*, 1909. (b) Wegmann's "The Design and Construction of Dams." (c) Data from Wm. Wheeler. Base thickness at 15 ft. above base. (d) *T. A. S. C. E.*, Vol. 53, June, 1904. (e) J. N. E. W. W. A., June, 1906. (f) Actually built but 30 ft. high. (h) 48 ft. below crest. (k) 94 ft. below crest. (l) Radius of center, both faces battered. (m) Temporarily finished to a height 22 ft. less. (n) *T. A. S. C. E.*, Vol. 67, 1910. (o) *E. N. R.*, May 2, 1918, p. 876. (p) *E. N.*, Oct. 12, 1916, p. 712. (q) *T. A. S. C. E.*, Vol. 83, 1919, p. 574. (r) *T. A. S. C. E.*, Vol. 78, 1915, p. 708. (s) *E. C.*, Feb., 1924, p. 340.

* For more complete statistics see "Masonry Dams," by W. P. Creager, 1917, p. 154.

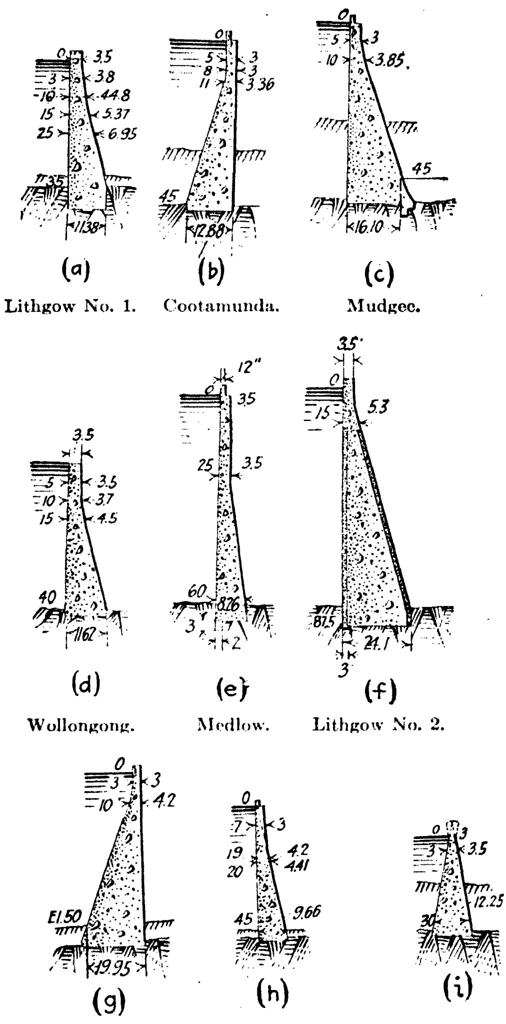
† Old dam; multiple-arch reinforced concrete dam, built in 1911. Placed at nearly 1000 by Davis.[2j]

‡ Computed from cylinder formula.

From Fig. 42, amount of masonry for an arched dam will be a minimum when the mean radius at any elevation is so chosen that 2θ is about 133°; the variation in masonry required for a given dam will be only about 1 per cent., provided that 2θ be held between 120° and 146°.* This method involves the independent determination of the dimensions of the successive arch-shaped slices between predetermined levels; each being considered primarily as an independent structure; such slices being thereafter superposed. At the top of the dam it will generally be best to choose 2θ near the upper limit (146°) for greatest economy, and at the bottom near the lower limit

* Old Bear Valley dam subtends but 53°.[2e] Flattening of the arch to prevent overhang of the top requires an enclosed angle between 140° and 60° for economy.

(120°) Slices of dam have no common center line; centers are located principally to get the length of arch as short as possible for a given distance across the canyon. Toward the bottom it is necessary to investigate, also, the maximum stress. The maximum arch compression will be $q \times \frac{2(R+t)}{2R + \frac{3}{2}t}$



(a) Lithgow No. 1. (b) Cootamunda. (c) Mudgee.

(d) Wollongong. (e) Medlow. (f) Lithgow No. 2.

(g) Tamworth. (h) Wellington. (i) Parkes.

FIG. 43.—Sections of curved masonry dams. Dimensions in feet.

and will exist along the downstream edge. The stress on the foundation does not need to be considered for dams less than 200 ft. high. This method can also be applied to the multiple-arch dam; the most economical result will be obtained by taking the top width equal to the distance between centers of buttresses, and bottom width equal to the distance between outside of buttresses, and by choosing the enclosed angles less than the most economical

at the top, where excess thickness must be provided for mechanical reasons, and gradually increasing this angle toward the foundation.

Synopsis of Recent Articles. William Cain: "The Circular Arch under Normal Loads," *T. A. S. C. E.*, Vol. 85, 1922, p. 233. Derives "nearly exact formulas for deflection . . . and formulas for moment, thrust, and shear, both for arches fixed (*encastré*) at the ends, and for those free to turn or 'hinged' at the abutments."

H. Hawgood: "Huacal Dam, Sonorá, Mex.," *T. A. S. C. E.*, Vol. 78, 1915, p. 564. Design and construction. Dependent on arch action for stability. Argument for cylinder formula. Table of principal arch dams.

L. R. Jorgensen: "An Example of Design of Arch Dam with Constant Angle," *J. Elec.*, Jan. 15, and Feb. 15, 1917, pp. 33, 110. "Improving Arch Action in Arch Dams," *T. A. S. C. E.*, Vol. 83, 1919, p. 316. Methods of grouting contraction joints discussed. Author has taken out U. S. Patent No. 1216234 (1915) on this method. "The Constant-angle Arch Dam, *T. A. S. C. E.* Vol. 78, 1915, p. 685. Features distinguishing from ordinary arch (cf. p. 137). Short-cut method for proportioning arch and cantilever stresses.

F. A. Noetzli: "The Relation between Deflections and Stresses in Arch Dams," *T. A. S. C. E.*, Vol. 85, 1922, p. 284. Method of calculating stresses resulting from deflections due to water pressure, change of temperature, shrinkage, lateral deformation, swelling, etc., which have been measured. "Arch Dam Temperature Changes and Deflection Measurements," *E. N. R.*, Nov. 30, 1922, p. 930. A suggested method of experimentation with full-size structures which may give information useful in design. "Gravity and Arch Action in Curved Dams," *T. A. S. C. E.*, Vol. 84, 1921, p. 1. Shows "some simple methods" for determining the proportion of the load carried by arch and cantilever action. "Simple approximate formulas" are developed for the determination of temperature and shrinkage stresses. Author's patent No. 1410217 (1922), applies to such a structure.

Parker: "The East Canyon Creek Dam," *T. A. S. C. E.*, Vol. 83, 1919–1920, p. 574. Design and construction of dam 140 ft. high above stream bed, with a center angle varying from 76 to 47 deg. The arch abuts against a wing of gravity section at one end, and of hollow deck type at other end. Dam was built during winter, and sand used was of doubtful quality. Computed by simple cylinder formula. Structure is successful.

Smith: "Arched Dams." *T. A. S. C. E.*, Vol. 83, 1919–1920, p. 2027. Method of analysis requiring "a considerable knowledge of mathematics," is offered, which avoids the obvious defects of the ordinary cylinder theory, *i.e.*, assumption that arch thrust is a maximum at the base where, as a matter of fact, due to foundation fixation, it vanishes.

HOLLOW DAMS

Use. In attempts to reduce the quantities of masonry and lessen unit foundation pressures, dams have been designed with buttresses or pier walls, supporting either arches (multiple-arch dams), or slabs (Ambursen type). Reinforcement is required in members subject to other than compressive stresses. Unreinforced dams have also been built with buttresses and arches designed as if in compression only, *e.g.*, Meer Alum dam in India, and Belabula dam in New South Wales. Various cellular dams, stabilized by ballasting the cells with water or other materials, have been patented. Most types of hollow dams have been patented, by Ambursen, Eastwood, Jorgensen (1915, No. 1282087), Edge, Turner, and others.

Disadvantages are lack of watertightness of thin sections, and their susceptibility to cracks which may expose the reinforcing steel to corrosion. In riot or war; such dams may be readily demolished. Skilled labor is required for the intricate form work. Although many advocates claim freedom from uplift, conservative practice considers the whole rock slab underlying the dam as subject in some degree to uplift.

Multiple-arch dams* have been widely used in Western† America and in Italy; they have been built as high as 228 ft.,[34] and one 279 ft. high is under construction for Suviana reservoir.[33] They should be restricted to solid rock foundations, or to low non-overflow dams where the foundations can be readily consolidated to eliminate settlement. They have been used on clay in Michigan.[2i] This type was preferred at Mountain Dell dam (Salt Lake City Water Supply), because of tendency to tighten under load; the seamy rock indicated probable leakage.[37]

Description. In general, multiple-arch dams consist of semicylinders, with axes inclined at 50° to 60° from the horizontal, arching between buttresses. Arches are in compression under normal loads. In recent American designs, each arch has a vertical axis for its upper few feet to do away with excessive secondary stresses. Vertical cylinders were used throughout in the earlier dams in India and at East Park reservoir, Cal.; in the latter, their use was incidental to enlarging the spillway capacity. For inclined arches, buttresses are spaced 25 to 50 ft. Buttresses and arch footings distribute the load to the foundation. Only on poor sites is it necessary to put in a continuous foundation slab under the buttresses. For a dam 120-ft. high, the buttresses and their supporting struts comprise 80 per cent. of the volume.[2e] Noetzli[35] proposes a saving in buttress material by using double walls with diaphragms between, claiming cost to be 50 per cent. of a gravity dam.

Advantages of multiple-arch dams are saving of masonry, lower cost and reduction of foundation pressures. Compared to a "gravity" dam, greatest economy is shown where material costs are high; when height exceeds 130 ft. there may be no saving (Jorgensen).[36b] Compared with flat-slab dams there are half as many buttresses, one-tenth the steel, and arches may be thinner than slabs (Blackwell).[36a] This type offers economy of 10 to 25 per cent. over the single arch below 125 ft. height, but above this the reverse generally holds true (McIntosh). Multiple-arch dams are not limited to narrow gorges with precipitous rock sides. Gate-house costs are reduced as valves may be housed beneath the slab.

Disadvantages, besides those mentioned above, are: Forms are more intricate than for flat-slab dams. Unyielding foundations are required as settlement of any pier would be dangerous. Unsuited to overflow section. Damage to thin sections by frost.‡ Blackwell[36a] fears that the failure of one section would wreck the whole structure. This did not follow at Gleno, see p. 145 (Authors).

Arch spans vary from 25 to 50 ft., dependent on the following conditions: All spans equal; theoretically the total concrete in the buttresses is independ-

* Compiled mainly from "Fundamentals in the Designs of a Multiple-arch Dam," by R. P. McIntosh, *E. N. R.*, Sept. 4, 1919, pp. 464–468. See also Janni, T. A. S. C. E. Vol. 88, 1925, p. 1142.

† See early paper by G. T. Dillman, *T. A. S. C. E.*, Vol. 49, 1902, p. 94.

‡ Gem Lake dam was rebuilt to a gravity section on this account; see E. N. R., July 2, 1925, p. 22.

ent of arch span; short spans save material, but this is offset by cost of forms and placing concrete in thin sections, as well as minimum thickness required for watertightness. High dams require the maximum span; this puts heavy loads on the buttresses, which have to be intricately supported against buckling.

Arches, in the earlier multiple-arch dams, were circular in a plane normal to the axis, and of uniform thickness—an uneconomical shape, subject to unnecessary secondary stresses, since crown is under less head than its corresponding haunch. McIntosh and others propose that the section in a horizontal plane be made circular and of uniform thickness as all parts of such a ring are under same water load. This results in an elliptical section (*i.e.*, practically three-centered), normal to the axis, wherein the thickness increases from the crown to the haunch. It is analytically demonstrable that line of pressure follows closely the center line of the arch.

Slope of water face was found to be 50° from horizontal for dams 50 to 75 ft. high, and nearer 60° for less heights, to produce minimum volume of dam, (45° has been used in Michigan.)[21]

Arch thickness can be found preliminarily by cylinder formula (see p. 136). (Ice and earth pressures are generally neglected, but were considered in the Suorva Lakes dams, Lapland.[72]) This method generally yields a thickness inadequate for stiffness, watertightness, and facility of construction, and designer must rely on judgment. To procure watertightness, the water faces are generally treated with gunite, or faced with rich mortar.

Central angle should approximate 133½° (see p. 139); a variation of 15 per cent. does not increase volume by more than 2 per cent. and is permissible to meet conditions.

Upstream radius is fixed when central angle and span are established. Keep this radius constant throughout. Radius is measured in same plane as central angle.

Arch Ring Analysis. The inclination of the arches makes stresses compressive under all conditions of water load, but tensile stresses caused by rib shortening and temperature changes require the use of reinforcement. To avoid confusion, "arch ring" will be taken normal to the oblique axis.

Ring. The extreme upper and lower ends of the arch barrel cannot be rigidly analyzed, due to foundation and crest influences. Weight of barrel is carried partly to the buttresses and partly to the foundation; the former is in ratio of $w \cos \theta$, where θ is angle of barrel axis with horizontal. This must be combined with the water load to get normal loading for the arch analysis. (Some designers add this to water load; others consider it acting normal to face of abutment, and use resultant.) The loads established, investigate the arch by the well-known elastic theory.* Be sure to include rib shortening and temperature effects in computing stresses. For a more complete discussion of this mooted question, see "Stresses in Multiple-arch Dams," by B. F. Jakobsen, *T. A. S. C. E.*, Vol. 87, 1924, p. 276.

Buttress Design. Establish trial dimensions from existing structures. Upstream face must be wide enough to accommodate the abutting arches. Method of tying reinforcement of arch into the buttress has been patented by Jorgensen. Adequate struts well distributed are needed to prevent buckling;

* See "American Civil Engineers Handbook," 4th ed., 1920, p. 737.

Table 45. Dimensions of Multiple-arch Dams, Ft.‖

Name	Tirso	Horseshoe	Mountain Dell	San Dieguito	Gem Lake	Eleanor
Maximum Height	220	210	145(100 –)	136	80	70
Clear Space between Buttresses	12.5	49	30.6	22.5	38.85	40
Buttress Thickness, Top	8.3	11§	4.4	1.5	1.85	5
Buttress Thickness, Bottom	23.33†	19.2§	8.0	4.0	4.25	8.92‡
Base Width	74.55	208		146	58.1	73.75‡
Buttress Reinforcing		18" Spacing of ¾" below 100 ft. ½" above 100 ft.		None		
Arch Ring, Shape		3-centered			Circular	Ellipse
Arch Ring, Extrados Radius		28.5	19.04		23.1	23*
Arch Ring, Rise of Extrados	⅓ Span		11.53	13.9		
Arch Ring, Central Angle		100°	120°	119° 57′		120° 50′
Arch Ring, Reinforcing			⅜"–24" c. to c. (V) ½" & ⅜"–18" c. to c. (H)	½"–36" c. to c. (V) ½"–12" c. to c. (H)	¼"–20" c. to c. (V) ½"–14" c. to c. (H)	
Crown Thickness, Max.	5.5	6.1	4.1	2.6	3.60	3.08
Crown Thickness, Min.	1.7	1.5	1.25	1.0	1.25	1.25*
Struts, Size	Arch bridges	1.25 × 1.25		4 × 4 (Tee shape)		
Struts, No. sq. ft supported by one		1000	580	1000		
Arch Barrel, Slope to Horiz.	57°	48°	10 on 12	45°	50°	50°
Concrete Mix, Arch			1:2:4	1:2:4	1:2:4	
Concrete Mix, Abutments			1:3:6		1:2½:5	
Engineer	Kambo			J. S. Eastwood	L. R. Jorgensen	O'Shaughnessy
Date	1923	1923	1918	1919	1916	
References	E. N. R. May 10, 1923, p. 820	T. A. S. C. E., Vol. 87, 1924, p. 359	E. N. R. Mar. 7, 1918, p. 455	E. N. R. Apr. 10, 1919 p. 720	T.A. S. C. E. Vol 81, 1917. p. 850	

* Horizontal Plane.
† Rough, pointed masonry.
‡ Below El. 175.
§ Outside of double walls which are 1.5 thick at top, 5.6 at base.
‖ Tidone Barrage, Italy, is 170 ft. high; see E. N. R., Oct. 29, 1925, p. 710.

this disadvantage of this type was met by Noetzli at Horseshoe[35] by the use of cellular buttresses. Stability of buttresses should be investigated graphically. Distribution of foundation pressures should be carefully analyzed to avoid crushing at toe. Methods are indicated by discussions in *E. N. R.*, Apr. 13, 1922, p. 623; June 15, p. 1009; July 13, p. 78; and Sept. 14, 1922, p. 451. In Michigan, spread footings have been employed on clay.[2i]

Failure of Gleno Dam.[38] On Dec. 1, 1923, a multiple-arch dam failed in North Central Italy. Height 143 ft., arch span 26.25 ft., center to center buttresses. Built on masonry base, as original plan called for gravity dam. Of 25 arches, 9 failed, together with buttresses. The failure is charged to shear of poor concrete in buttresses and inadequate design of base. Examination indicates the following faults: (1) failure to cut footings in rock for buttresses; (2) use of improper materials and lack of inspection of them; (3) poor mixing and lack of inspection of concrete; (4) use of unwashed aggregate; (5) lax inspection in pouring concrete; (6) failure to ram concrete in forms; (7) generally incompetent direction and supervision.

Flat-slab dams, evolved from the low wooden dams, consist of an inclined, reinforced-concrete slab supported by buttresses, beams and buttresses, or columns. Spans vary from 15 to 30 ft.—about 70 per cent. of those of multiple arches—so that more buttresses are required. Overflow or non-overflow sections may be designed. Some designs involve pre-cast slabs to obviate water-control difficulties during construction. High buttresses must be braced by struts. On poor foundations a base slab, properly vented to minimize uplift, can be used to distribute pressures. Dams of this type have been used on clay.[2j] Foundations are as important as with other types. Failures at Stony River, Plattsburg, and Austin, Tex. (see bibliography p. 146), are laid to the foundation rather than to the type of structure. J. K. Finch[40] points out the danger of sliding, which can be met by the use of cut-off walls such as were used to stabilize Stony River dam when reconstructed. At Cisco dam,[41] sliding on clay was prevented by a reinforced concrete cut-off wall tied into the face slab and ridges projecting 5 ft. below the base slab.

The Ambursen Company, New York, holds many patents and has constructed over 160 dams. Notable Ambursen dams are: Guayabal, Porto Rico, 115 ft. high and 2000 ft. long; Oklahoma City W. W.,* 55 ft. high and 1800 ft. long; Arkansas Light & Power Co., Hot Springs, 75 ft. high and 1000 ft. long; Estacada, Ore.[67] 76 ft. high and 405 ft. long.

Advantages. More economy and safety than in the "gravity" type for dams of low or medium height.[2e] Opportunities for flexible arrangements in stream control are a valuable advantage; numerous contraction joints divide the work into easy construction stages, reducing cost of plant and construction. Inclined deck utilizes stabilizing effect of the pressure, which, in a solid "gravity" dam with nearly vertical upstream face, acts wholly to displace the dam; ice thrust is eliminated by flat slope of deck; long base provides twice the safety against overturning, compared with "gravity" dam, and the safety increases with the flood height; statically determinate stresses[39] lessen waste of material due to uncertainties; uplift pressure eliminated by weep holes

* A spillway section designed for 15,000 cfs., safely carried 60,000 cfs. in October, 1923.

in the floor; accessibility for inspection; construction problems minimized as the type fits any foundation condition, and water control and river closure costs are small; construction time is 66 to 75 per cent. that of "gravity" dam (Ellsworth dam, 65 ft. high and 500 ft. long was built in 9.2 months); materials are 35 to 45 per cent. of "gravity dam;" costs 65 to 85 per cent. of "gravity" dam; tested by 21 years' use. Alterations in 1924 to dam at Ellsworth, Maine, showed the steel in the slab to be free from corrosion after 17 years.

Arguments against flat-slab type, for Black Canyon dam[68] by engineers of U. S. Reclamation Service: Drum gates, selected as the most suitable for passing the heavy ice flows, would involve complications and costly fastenings to the slab type; severe frost action* would injure thin, reinforced concrete; conditions were favorable for mass concrete.

Stresses, Ambursen Practice. Stresses in deck slabs, which are designed as simple beams, are based on 650 lb. per sq. in. compression in the concrete, 16,000 lb. per sq. in. tension in steel, and maximum shear in concrete, 60 lb. per sq. in. Concrete in buttresses is limited to a compression of 210 lb. per sq. in. and there is no tension.†

Bibliography of Flat-slab Dams

Stony River, W. Va. Construction, G. H. Bayles, *E. N.*, Jan. 22, 1914, p. 204. Failure, *ibid*, p. 211. Reconstruction, F. W. Scheidenhelm, *T. A. S. C. E.*, Vol. 81, 1917, p. 907. **Mathis Dams, Ga.** E. Lauchli, *E. N.*, Sept. 16 and 23, 1915. **Austin, Tex.** F. S. Taylor, *E. N.*, June 3, 1915, p. 1089. F. S. Taylor, *E. C.*, May 26 and June 2, 1915. D. W. Meade, *Report*, 1917 (see *E. N. R.*, Feb. 21, 1918). **Plattsburg, N. Y.** *E. N.*, June 8, 1916, p. 1106.

OVERFLOW DAMS: WASTE WEIRS

Forces Acting. Forces tending to destroy a waste weir or overflow dam are: (1) Water pressure due to (*a*) static head; (*b*) velocity of approach, considered as parabolic function varying from 0 at bed to 6 ft. per sec. at crest, and assumed to exert a horizontal pressure only; (*c*) buoyance (under base), considered to have parabolic distribution from 0 at toe to pressure due to full flood head at heel. (2) Silt pressure. (3) Ice thrust. (4) Wave impact. (5) Impact of floating objects. (6) Vacuum effect under falling water. (7) Frictional effect of overfalling water in contact with downstream face. (8) Wind pressure. (9) Crushing of foundation.[42]

Forces tending to hold the structure to position are: (1) Weight of masonry, acting by gravity alone. (2) Inertia of cross-section. (3) Silt and water-pressure on battered heel. (4) Weight of overflowing water. (5) Pressure of backwater on toe. (6) Cohesion of masonry. (7) Mechanical bond at base.[42]

Desirable features in a spillway dam and channel are: (1) downstream face a smooth ogee, or else so stepped that water is broken by falling from step to step.‡ (2) Straight plan.§ (3) Unbroken crest, shaped to conform to

* A flat-slab dam built in 1917 in the high elevations of the Sierras was so badly damaged by frost that it had to be converted into a semigravity type.

† For design methods, see Beardsley, *E. N.*, Apr. 23, 1908, p. 452.

‡ See last paragraph, p. 147.

§ S. H. Woodward designed Cherokee dam curved in plan to fit rock foundation economically and to increase spillway capacity so as to discharge 200,000 sec. ft.

the lower nappe for the maximum flood. (4) Easy curve joining downstream face to floor of channel, or a receiving pool of adequate depth. (5) Channel width same as spillway for a distance sufficient to permit the full development of the normal jump for ogee face. (6) Floor of channel near dam nearly level throughout. (7) Water taken in a single fall along the entire crest. (8) Proportion channel to keep the jump near toe of dam.[45]

Vacuum Effects on Ogee Face. According to G. S. Williams, experiments on a small model showed reduction of atmospheric pressure of one-sixth atmosphere (about 2.5 lb. per sq. in.) at the toe of an overflow dam, due to partial vacuum under the falling water. Tests were on a model 8 ft. high, with 2 ft. of water on crest. He estimated that this pressure might equal 12 lb. per sq. in. for a high dam.[42] The partial vacuum between water sheet and face of Sudbury dam engendered by an 11-ft. fall, sets up atmospheric waves that rattle windows to a disturbing degree a mile distant.[43] Air is supplied to downstream face of an ogee type dam built by Tacoma Light & Water Co. across Green River, Wash., by 8-in. sewer pipes 20 ft. on centers extending from apron to air supply pipe, parallel to crest, leading to atmosphere at abutments. Maximum section of dam is 16.13 ft. high, with base width 24.10, designed for a factor of safety of 2.73 under 15-ft. flood head. Length of overflow, 150 ft.[44] Like provisions in Pedlar River dam, p. 171.

Design methods follow in general those outlined on p. 132. The first step is a tentative design for shape of crest and downstream face. Precedent as exemplified on p. 168 to 187 is helpful. For a mathematical discussion of the shape, and for design methods, the reader is referred to "Engineering for Masonry Dams," by W. P. Creager (John Wiley & Sons, Inc., 1917). It is essential to reduce vacuum effects and so to shape the "bucket" that the discharge of water from the foot of the dam is facilitated. On Holyoke dam (see p. 171) with a 4-ft. head on crest, the falling water is in contact with the face from crest to toe. This is true up to a head of about 5 ft., beyond which the surface of the sheet becomes so rough that it is not possible to investigate the contact. Where the foundations are questionable, Sickman advocates horizontal aprons tangent to face curves at lowest points as better than "uplift," to give protection against erosion beyond the apron. Uplift causes water to strike another blow. To restrict overflow so that the ends of the dam would not be endangered, diversion walls converging from 485 ft. spacing at crest to 185 ft. at "bucket," were built on the downstream face at Eel River dam.[47]

Meyer[60] investigated the path of the water on an ogee face and found that where the backwater is lacking at the foot—the general case—there will be no body of water to absorb the velocity of the rapidly moving sheet. He proposes the usual crest shape, and the provision of a hollow bucket to provide sufficient back pressure to absorb the excess energy. (Patents applied for).

To avoid the excessive velocities at the base of the face, some engineers prefer "pounding out" the energy by a series of steps. Pedlar River, Gilboa, Crosswood and Roseberry in Scotland, and Urft in Germany are among the dams with stepped spillway faces. No principles for their design appear to have been formulated; at high heads (great depths on crest), the velocity of approach sends the water out to form a nearly uniform sheet rather

than a series of cascades. At low heads, the designer may investigate each step as an independent drop. German engineers use Kutter formula.[45b] Continual impact on the steps is hard on materials, but the wear depends upon constancy of overflow, as in a river where storage behind dam is relatively small, or the infrequency of overflow, as in a small stream where storage is relatively very great. At Gilboa the steps are faced with native sandstone. Tests were made on small-sized models in connection with this design.[45c]

Water Cushions for Overflow Dams.[48] Width of floor of cushion depends somewhat on section of dam, but need not exceed $8\sqrt{D_w}$, and should not be less than $6\sqrt{D_w}$. For dams with a vertical drop and a horizontal apron on same level as toe of dam, this masonry apron should be formed of fairly regular blocks of stone or Portland cement concrete, the width varying as just given, and the thickness or depth of stone from $\frac{1}{5}(H + D_w)$ to $\frac{1}{4}(H + D_w)$.

$$D = C\sqrt{H^3\sqrt{D_w}}.$$

D = depth of cushion below top of subsidiary weir, see Fig. 44.
C = coefficient, dependent upon material used for floor of cushion; it varies between 0.75 for compact stone, and 1.25 for moderately hard brick;
H = height of fall, from surface to surface;
D_w = maximum depth of water to pass over crest.

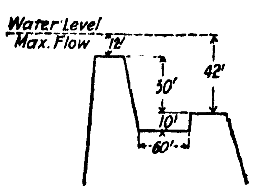


FIG. 44.—Periyar dam waste weir.

Greatest depth of hole formed by the waterfall in the new outlet to Mudduk Masur Tank (reservoir) is 24 ft. In ordinary stages of the river, the general depth of the cushion is to the height of fall as 3 to 4 where greatest action occurs, and 1 to 2 in other places.*

An experimental fall on Bari Doab Canal (India) had a height of 6.9 ft. and a depth of well of 9.0 ft. and 3.6 ft. on crest, which gives depth of well to height of fall as 3 to 4. The water had no injurious effect on bottom of well.

Periyar dam (India) waste weir. Head on subsidiary weir one and one-half times head on main weir. Fall from surface to surface varies from 24 to 30 ft., and depth of cushion from 10 to 28 ft. (See p. 182 for section of Periyar dam.)

The materials of an overflow dam and of the channel below it are subject to the extremely erosive actions of water, floating ice and logs, and should be selected and placed to afford maximum resistance.† Wigmore reports green concrete 3 days old submerged by a flood 5.5 ft. deep[49] over the uncompleted dam without injury; not more than 2 in. was washed off the top. According to Lauchli,[50] the following defects in construction methods assist erosion: (1) leaky forms; (2) planes at end of day's work; (3) horizontal lift joints; (4) vertical joints at butting surfaces of forms. Slight surface irregularities always furnish points of attack.

* Holes of appreciable depth have been worn by the overfall at both Keokuk and Hale's Bar dams.

† See also p. 797.

DAMS AND WEIRS ON PERVIOUS FOUNDATIONS

Dams on pervious foundations are generally *low diversion dams* which turn water into a canal. In India, height seldom exceeds 12 ft. from river bed to crest; some American dams exceed 20 ft. The pervious substrata render the sites unsuitable for storage. Among the types developed for such sites have been all the forms of hollow dams* as well as the types used in Indian irrigation practice (Figs. 45, 47, and 52, and described below). Diversion weirs have been founded on sandy bottoms also in Egypt and in Western United States

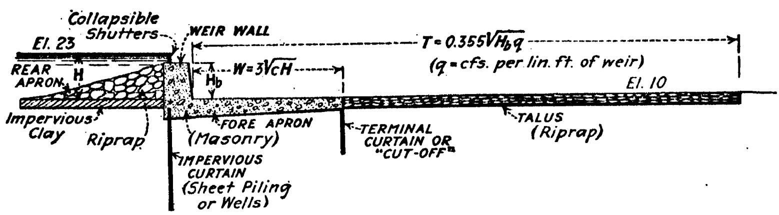


FIG. 45.—Narrora type of weir on pervious foundations. Parts and dimensions.

(Fig. 53). The Ohio River Board suggests founding on piles.[51] Names of the parts of a diversion weir are given in Fig. 45.

Destructive forces, peculiarly menacing in this type of dam, are: (1) Percolating water in pervious substratum. (2) Upward hydrostatic pressure on thin fore apron† (see also p. 119). (3) Impact of overflowing water. "In spite of destructive influence of a large river in high flood and erosive action on its sand foundations, it is quite practicable to design a work of such outline that

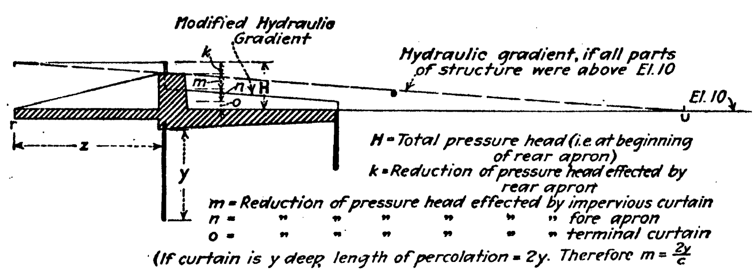


FIG. 46.—Influence of component parts on hydraulic gradient. (Same structure as Fig. 45.)

it will successfully remain as solid and permanent as one on bed rock" (Bligh). A weighty structure on a permeable, unstable substratum to which water has access, resists these hydraulic forces in two ways: Compression and retention of sand in situ retards erosion; weight of structure resists uplift. Impact of overflowing water is resisted by water cushion and its erosive action by fore apron and talus.

Percolation Factor. If velocity of percolating water can be sufficiently reduced, scouring or undermining will not occur. (For lists of scouring

* The requirements of a broad base makes one of the hollow types with foundation slab particularly suitable.

† Colman [13a] found that physical characteristics of porous substratum have little influence on the upward pressure. Hydraulic jump may add to the uplift tendency. See *E. N. R.*, Apr. 25, 1918, p. 831.

velocities, see p. 106.) Velocity of percolating water = constant × slope. (See p. 78). We can write: Safe velocity = $K \times H \div L$. Or $L = \dfrac{K}{\text{safe velocity}} \times H$. Substituting C, called "the percolation factor," for $\dfrac{K}{\text{safe velocity}}$ we get Bligh's equation: $L = CH$. Values of C are naturally dependent on quality of sand; those in Table 46 are based on practice, chiefly in India and Egypt; for method of fixing, see p. 155.

Table 46. Values of Percolation Factor, C, for Dams[74]

Material	Class	C
Light silt and sand (60 per cent. passing 100-mesh sieve) as in Mississippi and Nile rivers	I	18
Fine micaceous sand (80 per cent. passing 75-mesh sieve) as in Colorado and Himalayan rivers	II	15
Coarse-grained sands, Central and South India*	III	12
Boulders or shingle and gravel and sand mixed	IV	9 to 5

*Most river beds belong to this class.

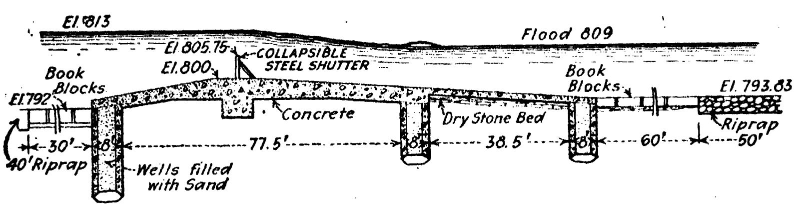


FIG. 47.—Merala weir.
(Bligh.)

Types. Figure 52 represents the annicut or rapids type. Some Madras annicuts are over 1500 years old. Both upstream and downstream aprons are riprap. A few masonry cut-off walls in the downstream apron prevent displacement of rubble. Best known weirs of this type are: Okhla weir on Jumna River (Fig. 52); Kistna, built in 1849; Dehri weir, on Son; Madaya

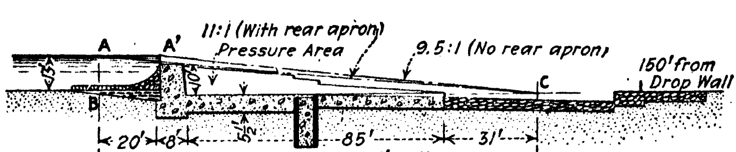


FIG. 48.—Narrora weir. Original section.

weir, Burmah; Laguna weir, U. S. Reclamation Service. The theory developed for dams with impervious floors (see p. 153) applies likewise to pervious floors. The annicut type is adaptable to the finest sands. Where rough materials and unskilled labor are plentiful, there is economy in this type; the only masonry in mortar is in the weir wall and cut-off walls. Maintenance cost is excessive due to settlement of fore apron. In 20 years, Godaveri weir has required for maintenance nearly as much material as first used. The fore

apron encroaches on the waterway of overflowing water, causing higher velocities, and greater tendency to erosion. Okhla (Fig. 52), has had velocities as high as 18 ft. per sec., measured 20 ft. downstream from weir wall. Same flood would cause 8 ft. velocity in Narrora type.

A second type is represented in Figs. 45, 48 and 49. The design of the low weir wall itself is of minor consequence; the important part is the apron. Figs. 53 and 57 are American examples of this type.* The weir wall of Granite Reef weir is weaker than in Indian designs (Fig. 48), but the fore apron is stronger. For hard-pan, clay, or firm gravel, this type is suitable. It is expensive to build, on account of quantity of masonry in weir wall and fore apron, but is least costly to maintain. A large waterway for overflow is provided. In some dams, waterway has been sacrificed to save masonry by laying fore apron on river bed, thereby decreasing the bursting pressure on it, utilizing the unsubmerged weight of the materials to resist the worst condition of bursting pressure (*i.e.*, no water flowing over to offset upward pressure), and doing construction work in the dry. Cost of unwatering and scarcity of materials near site made Narrora weir, of this type, cost twice Okhla weir (Fig. 52), which is of the first type.

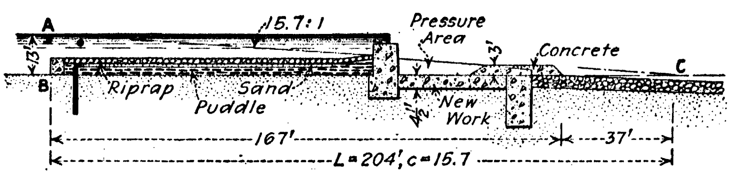


FIG. 49.—Narrora weir, after repairs.

Merala type (Fig. 47), is adaptable to finest sands; it offers little obstruction to passage of flood waters, but restricts channel, just as Okhla type does. As most of the fore apron lies close to hydraulic gradient (compare Fig. 46) the thickness of the expensive part can be reduced, as explained in next paragraph. Also some wet construction can be eliminated. This is the usual type in Punjab and North India (Parker).

Hydraulic gradient is shown in Fig. 46. Ordinates below hydraulic gradient represent pressures on structure below. Gradients shown in Figs. 46, 48, 49, 53 and 55 are not exactly true, as no allowance for entry loss has been made. This loss is considerable (see "The Action of Water under Dams," by Colman, *T. A. S. C. E.*, Vol. 80, 1916, p. 421.) Since $L = C \times H$, $H = L \div C$. To compute the neutralization of head due to traveling a distance $= 2y$, divide by C. In Fig. 46, $m = 2y \div C$. Likewise, $k = z \div C$. Were there no barrier to water passage but a vertical wall at r and flooring at elevation 10, flooring length L (*i.e.*, length of enforced percolation), must be $C \times H$, extending to U. Obviously, we can provide substitutes for this excessive horizontal distance. This has been accomplished by use of sheet piling, or wells. Tests in India have shown that water will follow down and up the sides of an impervious vertical obstruction, traveling a distance $2y$ to pass an obstruction of depth y,

* American practice retains the ogee face, which has been discarded in India because of high velocity with which water leaves it.

so that value of vertical obstruction is twice that of an equal horizontal length.* This does not hold good if two cut-offs be spaced closer than twice the depth.

Rear Apron. As shown in preceding paragraph, rear apron can be as effective in cutting down bursting pressure on fore apron as is piling. The rear apron must be impervious to enforce percolation beneath it; there must

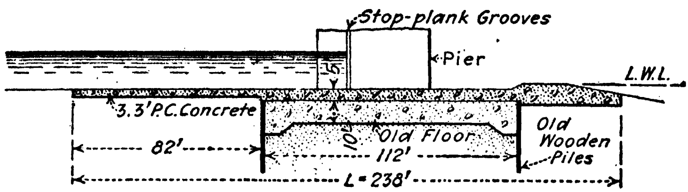


FIG. 50.—Repairs to Grand Barrage.

be a watertight connection with the weir wall. Imperviousness can be gained by use of a clay bed or silted riprap. Since there is no unbalanced pressure, materials need not be of the best; nevertheless, practice calls for not less than 4 ft. thickness. Top of apron should be protected by paving against scour by velocity of approach. If rear apron fails, as at Narrora, undesirable

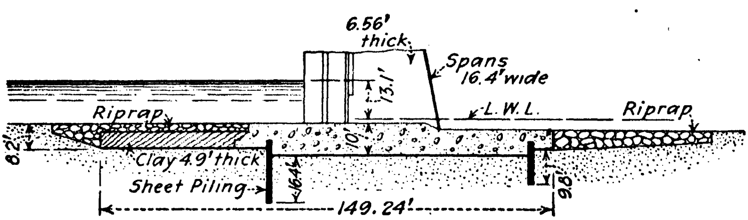


FIG. 51.—Zifta regulator.

pressures, perhaps full head H, are put on the fore apron, and it may blow up. Efficacy of the rear apron in reducing pressures on the fore apron is shown in Fig. 49, Narrora weir after repairs. The virtue of a rear apron was shown in Egyptian barrages. Figure 50 shows repairs to Grand Barrage; foundations were too weak for additional sheet-piling, apron was superposed on old floor,

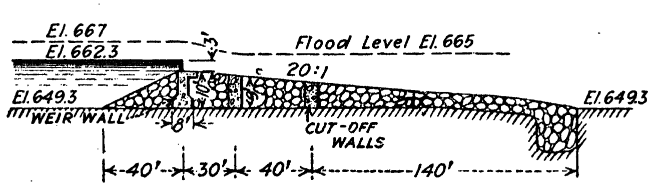


FIG. 52.—Okhla weir.

and whole work grouted; 13 ft. head of water was held back after repairs; barrage was useless before. The apron was designed on theory that the effective weight of the curtain when submerged should be sufficient to neu-

* In his earlier studies in "Practical Design of Irrigation Works," Bligh assumed that water traveled a 45-deg. path through the sand to reach the bottom of the obstruction. Later tests caused him to modify his view. Tests by Miami Conservancy District[52] indicate that $2y$ is too large a value, although not greatly in error.

tralize the uplift completely. Uplift was estimated from Okhla data. Ignoring effect of the old wooden piling (as probably permeable) the ratio of enforced percolation L to head H, = 238 ÷ 13, or about 18. This corresponds to value of C assigned in Class I, Table 46, and gives an inkling to process of fixing these values. In Zifta regulator (Fig. 51), tight piling makes total L = 215 ft. Head being 13.1, C = 16.4. In Assiut regulator, L = 241, H = 11.5, so C = 21. Level of rear apron is not dependent on that of fore apron, although they are often put at same elevation. Rear apron at crest level interferes with discharge.

Curtains, or **sheet piling,** also have their influence in cutting down pressure on the fore apron. Local economics must dictate depth of these curtains. For each case, designer must balance reducing depth of piling against extending rear apron upstream. Colman's test on a small scale model[13] led to following conclusions: (1) Piling at heel of dam reduces pressure on floor, and piling at toe increases it; (2) piling at heel, to be effective, must be tight, small leakage destroying its action; (3) piling at toe should be loose, to prevent

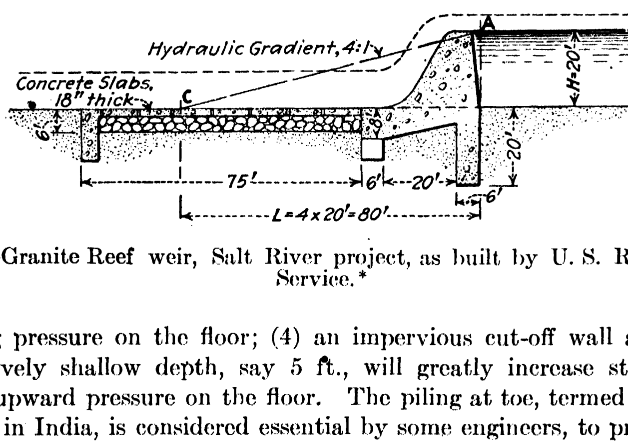


FIG. 53.—Granite Reef weir, Salt River project, as built by U. S. Reclamation Service.*

increasing pressure on the floor; (4) an impervious cut-off wall at heel, of comparatively shallow depth, say 5 ft., will greatly increase stability by reducing upward pressure on the floor. The piling at toe, termed "terminal curtain" in India, is considered essential by some engineers, to prevent displacement or undermining of fore apron, in case talus is washed away. Parker[76] questions the efficacy of deep piling, believing that apron construction is better than piling, for neutralizing head, but admits that it cannot be proved. Prior to 1870, no weir was built without a curtain of wells of square blocks.† Bligh says: "Although their value from hydrostatic point of view was not known, it was felt they were an indispensable adjunct." When Okhla weir (Fig. 52) was built without curtains, precedent was violated. Failure was predicted. This has not happened; 13 ft. head is stored.‡

Fore apron protects the bed from scour of overflowing waters; it must resist uplift from water percolating beneath dam. The length is a matter of judgment. Based on Indian practice, Bligh proposes as least length to give protection against erosion: $W = 3\sqrt{CH}$§ (see Fig. 45). Parker suggests

* Floods damaged the fore apron in an area 75 by 540 ft. wide, and concrete 6 ft. deep was substituted.[53]

† These would be termed "cut-off walls" in American practice.

‡ Okhla weir passed a record flood in Sept, 1924 and is considered "perfectly secure." (Correspondence with Dept of Industries and Labour, Government of India.)

§ "Cyclopedia of Civil Engineering," 1916. In 1907, in "The Practical Design of Irrigation Works;" also in *E. N.*, Apr. 13, 1911, Bligh proposed $W = 4C\sqrt{H + 13}$. The revised formula gives materially smaller values for W with same values of C.

$W = 3H$ in clay or weak rock. With no flow over weir, fore apron is under unbalanced hydrostatic pressure. This upward pressure is resisted by the weight of the material. Since fore apron is generally submerged, submerged weight must be used in calculations. In Fig. 46, fore apron at foot of weir wall has to resist an uplift $= H - (k + m)$, nearly. If expressed in pounds, this uplift per sq. ft. $= 62.5\ (H - (k + m))$. Weight of submerged material,

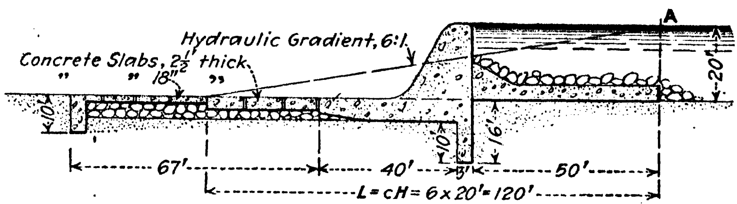


FIG. 54.—Granite Reef weir, as proposed by Mr. Bligh.*

t ft. thick $= t\ (p - 1) \times 62.5$ per sq. ft. p = specific gravity of the masonry, generally 2.5. Equating these: $t = (H - (k + m)) \div (p - 1)$. It is customary to make weight one-third greater than actual requirements, so $t \doteq 4\ (H - (k + m)) \div 3(p - 1)$. Parker incorrectly figures hydrostatic uplift = ordinate from hydraulic gradient to *bottom* of apron. Fore apron can be given trapezoidal section corresponding to diminution of water pressure, as in Fig. 45, to save materials. Bligh advocates minimum thickness of 3 ft. Etcheverry[77] recommends at least 1 foot of concrete for low falls; 2 ft. for falls of 20 ft. or more.

Any extension of the impervious fore apron beyond actual necessities is to be deprecated as increasing hydrostatic pressure. On bowlder bed, the fore apron can be made pervious by leaving spaces between the blocks of

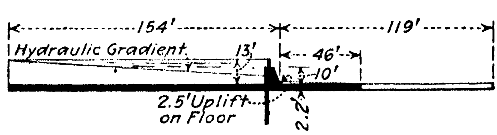


FIG. 55.—Design on a Class I sand.

concrete of which the floor can be built. This will necessitate a corresponding increase in length of rear apron to compensate for reduction in length of enforced percolation, so that it is of doubtful utility. With sand foundation, such construction would not be practicable, as sand would be carried through the open spaces. In some weirs the fore apron is raised above low water level to avoid wet construction; riprap (talus) then will slope down to that level or below.

Failure and repair of Narrora weir at head of Lower Ganges Canal, India. Figure 48 is section, as originally constructed. Ratio of head to length of enforced percolation, including effect of circular undersunk wells considered tight, was 144 to 13 or 1 on 11. If fore curtain was not tight, the gradient was

* Letters patent (914,559) for a similar structure were issued to W. G. Fargo in 1909

steeper. The structure stood intact for 20 years, always in unstable equilibrium. The fore apron must have been in high tension. Shortly before failure, borings revealed the fact that the floor was entirely undermined, and was supported against collapse only by hydrostatic pressure. After a heavy freshet in which the rear apron was washed out, the hydraulic gradient steepened to 1 on 9.5, and 350 ft. of floor was blown up (March, 1898). Repairs, (Fig. 49), lengthened the rear apron to 80 ft.; weir floor was built thinner, of heavier material, and weighted by mass of material above original floor level, so that its weight in air was effective in resisting worst condition of uplift. The length of the impervious fore apron was reduced, thereby decreasing uplift. Percolation factor in new structure was increased to 15.7. Experience with Egyptian barrages and Narrora weir shows that a weak structure can always be strengthened by extending rear apron.

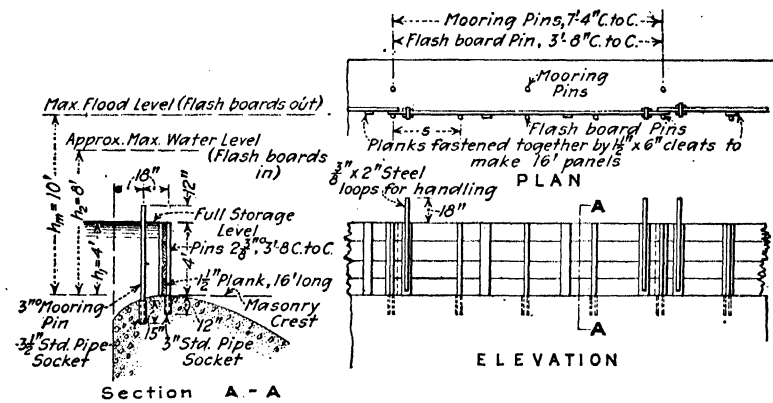


FIG. 56.—Typical flashboard (shutter) installation.
(From "Engineering for Masonry Dams," by W. P. Creager, page 224 (J. Wiley & Sons, Inc., 1917).*)

Chenab River weir was built on same class of sand as Narrora. After years of insecurity it collapsed by undermining, with gradient 1 on 8.3. Repairs lowered gradient to 1 on 16.

Talus. Beyond the impervious fore apron, riprap, or stone pitching, is necessary to prevent erosion by overflowing water. Bligh proposes as distance to end of talus from toe of weir wall, an empiric formula† based on Narrora conditions.

$$T = 10C\sqrt{H_b \div 10} \times \sqrt{q \div 75} = 0.367\sqrt{H_b q} \quad \text{(for Merala type, use } 0.39C\sqrt{H_b q}\text{)}.$$

H_b = height of weir or drop wall above fore apron (10 ft. at Narrora); q is discharge over weir in cfs. (75 cfs. at Narrora). Table 47 shows consonance of this formula with practice. On clay or weak rock, Parker recommends $T = 6H$. On other foundations, he uses Bligh's formula. In case talus lies within area of enforced percolation, as happens in Okhla type, uplift is resisted by weight of riprap, corrected for submergence, where necessary, and

* See *Ibid.*, p. 225, for derivation of diameter of pins, stressed to say two-thirds of elastic limit, when pond is at full storage level.

† Based on theory that T will vary with square root of height of obstruction above low water, with square root of unit flood discharge over weir crest and with C.

allowing for void spaces. The talus should be 2 to 6 ft. thick, a matter of judgment according to materials. Continual maintenance is necessary. If ogee type of overfall is used, Parker recommends a paved talus of closely fitted concrete blocks as at Granite Reef.

Table 47. Actua and Calculated Distances of End of Talus from Toe of Drop Wall[61] (Bligh)

River	Name of weir	Type	C	H_b†	q‡	Distance, Ft.	
						Calculated	Actual
Ganges......	Narrora	B	15	10.0	75	150	140–170
Coleroon	Coleroon........	B	12	4.5	100	92	72
Vellar.......	Pelandori........	B	9	11.0	100	108	101
Tampraparni	Srivakantham....	B	12	6.0	90	102	106
Chenab......	Khanki..........	Al	15	7.0	150	182	170
Chenab......	Merala..........	Al	15	7.0	150	182	203
Thelum......	Rasul...........	Al	15	6.0	155	160	135
Penner......	Adimapali.......	Al	12	8.5	184	172	184
Penner......	Neffore..........	Al	12	9.0	300	228	232
Penner......	Sangam.........	Al	12	10.0	147	168	145
Godaveri. ...	Dauleskeviam....	Al	12	13.0	100	158	217
Jumna.......	Okhla...........	A	15	10.0	140	210	210
Kistna.......	Beswada........	A	12	13.0	223	236	220
Son.........	Dehri...........	A	12	8.0	66	100	96
Mahanadi....	Jobra...........	A	12	100.0	140	163	143
Madaya.....	Madaya.........	A	12	8.0	280	207	235
Colorado.....	Laguna.........	A	15	10.0	Below Min.	140	200

† Feet. ‡ Cfs.

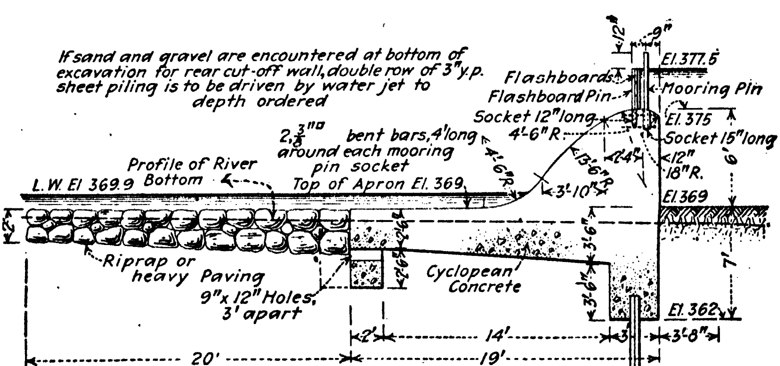


Fig. 57.—Design of dam for Burmus Paper Co., Inc., West Dudley, Mass.
(John H. Gregory, Consulting Engineer.)

Shutters. Most weir crests are provided with collapsible crest shutters, often 3 to 6 ft. deep. This enables the permanent crest to be kept low, offering less obstruction to floods. Shutters increase the static head, maximum being when upstream water is at shutter crest, downstream channel empty (Fig. 56).

Procedure in Design. Given head, $H = 13$ ft., and sand at site of Class I (Table 46); Length of enforced percolation, $L = C \times H = 18 \times 13 = 234$ ft. This must be made up by fore apron, piling, and rear apron. Length of fore apron must be $= 3\sqrt{CH} = 3\sqrt{18 \times 13} = 46$ ft.; 18 ft. sheet-piling is available; percolation length = 36 ft. Length to be made up by rear apron = $234 - 36 - 46 = 152$ ft. (Generally, local conditions will dictate economy in long piles or in long rear apron.) Distance to end of talus, if $H_b = 10$ and $q = 150$, = 254 ft. Maximum uplift on floor $= 13 - \dfrac{152 + 36}{18} = 2.56$ ft. (Fig. 55). $t = \dfrac{4}{3} \times \dfrac{2.56}{2.5-1} = 2.3$ ft.

Earth Pressures. Distribute load of structure so as not to overload the foundation. Parker suggests as safe: 1 ton on fine Nile silt, 2 tons on coarse sand, 4 tons on clay.

Bibliography, Dams on Porous Foundations

A. L. Harris: "The Diversion of Irrigating Waters from Arizona Streams" *T. A. S. C. E.*, Vol. 77, 1914. W. G. Bligh: "The Practical Design of Irrigation Works," 1907 (Van Nostrand); "Cyclopedia of Civil Engineering," 1916, Vol. 9 (Am. Tech. Soc.); *E. N.*, Dec. 29, 1910, and April 13, 1911. Parker: "Control of Water," 1913. Buckley: "Irrigation Works of India," 1905. B. A. Etcheverry: "Irrigation Practice and Engineering," Vol. 3 (1916) (McGraw-Hill Book Company, Inc.) S. V. Kanagasabai Pillai: "The Design of Masonry Weirs," 1920, the Author, Madras. Blanchard: "Floating Dam on Gila River," *E. N. R.*, Nov. 16, 1922, p. 824. "Repairing Laguna Dam on the Colorado," E. N. R., Dec. 18, 1924, p. 1002.

CONSTRUCTION

On this large and special subject but a few notes can be given. The reader is referred to various books, notably "The Construction of Masonry Dams," by Chester W. Smith, 1915, and "Irrigation Works," by A. P. Davis, 1917. The chief concerns of the designing engineer are water control, procuring of materials of construction and laying of masonry in such an order that cracks are minimized. For "Foundations," see p. 125.

Openings through dams near the base for temporary tunnels for the stream or roads or permanent large pipes cause undesirable concentrations of pressures, where pressures are naturally of maximum value. These openings should be carefully looked after in design, construction, and maintenance.

Water Control during Construction of Dam. To dispose of the stream or other water occupying a dam site much expense and ingenuity are often demanded. A stream may be diverted through tunnel, flume, pipe, or temporary channel, or may be carried across the foundation pit in pipe or flume. Quantity of water, character of stream, and nature of site will dictate the method. Cofferdams are usually required in connection with carrying the stream across the pit, and diversion dams for most schemes of diversion. Pumping is always necessary to care for ground water, and leakage past the temporary dams, and from the pipe or flume. Sometimes a succession of cofferdams is used, each unwatering a portion of the site and forcing the stream into the remaining portion of its channel. Naturally seasons of low flow are chosen for such operations. In the masonry first built, special devices are generally provided for making a quick closure of the passage or passages finally left for the stream. Simple, strong, iron, or wooden sluice, or flap gates are often used for the first step and are then quickly backed with masonry, the remainder of the filling being built at convenience. Provision should be made for grouting

around this plug to fill shrinkage spaces; plug should be bonded to sides of passage. Stream closures have been made by lowering caissons into the place, notably at Spanish River, Ontario.[5] Sunken barges have been employed as a nucleus for low dams on the Allegheny River.

Cross River Dam. Two temporary dams were built, above and below the dam site, and paralleling the proposed dam. These were connected by 560 ft. of wooden flume, 16 ft. wide and 6.5 ft. deep, and two 5-ft. steel pipes, extending across the dam excavation, supported on steel bents, and enveloped by the dam as it rose. When the time came to fill the pipes with masonry, a wooden bulkhead was built across the flume just above the pipes, and rubble, rather than concrete, was placed in successive walls, about 2 ft. thick. At the top, spalls were rammed into fairly dry mortar to close the pipe as tightly as possible; grouting was done through $1\frac{1}{2}$-in. wrought-iron pipes, placed 3 in.

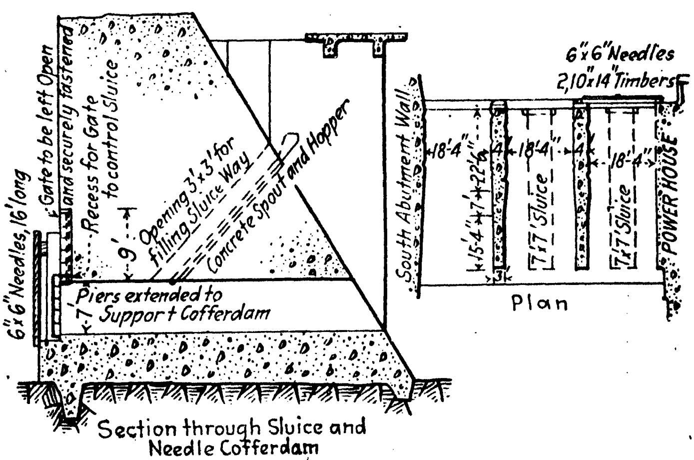


FIG. 58.—Stream-control opening in Brule dam, closed by needles.[64]

below the top. Fifty-eight bags of cement were used for grout in one pipe, forty-eight in the other.

Laying Masonry. For economy, safety and convenience, with few exceptions, 8 tons should be the maximum weight of stones or blocks used; up to 16 tons may be handled with unusually large equipment. If a good quarry is available near the dam site, coursed or random ashlar masonry can be furnished and built into the dam almost as cheaply as concrete blocks; a long haul might cast the balance on the side of the blocks. For massive work, involving heavy moldings or deep coves, concrete blocks would effect economy. It is cheaper to do cutting when the concrete is green, but preferable to wait until it has hardened.

Face joints in masonry should be horizontal if the batter is steep, but may be normal to the face if the slope of the dam is flatter; however, the contractor claimed that the latter proved expensive at Wachusett dam. Joints should be filled full to the face when the stones are laid, and properly finished, preferably concave. Raking out joints and repointing at a later date does not give such good results as the pointing mortar frequently comes out. No case is known of stones having spalled when joints were filled to the face when the masonry

was laid. Retaining walls in Philadelphia, built by the same contractor under two engineers, one of whom repointed, and the other finished joints as laid, illustrate this point. Within 1 year after completion, the portion repointed had suffered from frost about 25 per cent., due to mortar being forced out; at last reports, the other portion was in excellent condition.

On the upstream face of Wachusett dam, in addition to ordinary headers, (every third stone in every second course), a continuous course of headers was laid every 20 ft. to support the stones above; under these courses the joints were left open until the dam was built to full height and the rubble interior had had a chance to compress. Headers were used in the downstream face with the same frequency as in the upstream, except that there were no continuous courses. Face joints were raked out and not pointed until the dam was nearly completed.

Cyclopean masonry* is more economical than coursed masonry where good rock is not available, although it requires more cement. Placing cyclopean concrete requires use of pre-cast face blocks, cut stone, or face forms, as well as cross walls built up in advance to restrict the area of concreting. Racking back, *i.e.*, irregular stepping away from a vertical cross plane as the masonry rises, produces a tendency to unsightly cracks. Generally, concreting is done consecutively between contraction joints. In bonding new work to old on a horizontal plane, wire brushing may be employed, or embedded rocks left projecting above the contact surface. Experience with the latter on Wanaque, N. J., core wall is that temperature movements are restricted, and there is a tendency for cracks to develop. Form work may be saved by use of cantilevered studding, anchored at lower end by bolts left in the masonry. A sliding arrangement reduced handling at La Loutre dam,[65] and on concrete core walls in earth dams at Ashokan reservoir.

Rate of Laying Masonry. On O'Shaughnessy (Hetch-Hetchy) dam, average for 16-hr. day in May, 1922, was 1600 cu. yd.[56] At Elephant Butte,[57] 380,200 cu. yd. were placed in year ending Mar. 1, 1915. At Kensico,[58] 489,800 cu. yd. were placed in 8.5 months (Mar. 17–Dec. 2). At Arrowrock Dam, 2174 cu. yd. was average per 16-hr. day in June, 1914.[59]

Concrete blocks for facing provide a satisfactory substitute for forms, and improve the appearance. Casting can be begun at a near-by yard while foundations are being dug so that time is saved. Use a mix of $1:2\frac{3}{4}:5\frac{1}{4}$ to $1:3:5$. The best finish is obtained by casting the blocks with faces to be exposed down and against a steel plate in the forms. For Cross River dam, concrete weighed 152 lb. per cu. ft.; the largest blocks contained 87.1 cu. ft., and weighed 6.62 tons. Three months seasoning should be allowed; this fact is demonstrated by the high percentage of breakage after shorter seasoning; 4.7 per cent. of the blocks were rejected. The blocks constituted 11 per cent. of the total mass. Those blocks placed in the zone from 6 in. above full reservoir level to a few feet below have been affected by the action of the elements so that the original smooth surface has been destroyed, but this injury is superficial and negligible. A $1:2\frac{1}{3}:4\frac{2}{3}$ mix was used. Similar blocks were used on Cumberland, Croton Falls, Olive Bridge, Kensico, and other important dams, for a large part or the whole of one or both faces. Rejections on the latter dams were a very small percentage. The course height was commonly

* See also p. 802.

2.5 ft. Blocks were used also for one face of each expansion joint in some dams. One or two courses were laid in advance of the cyclopean or other hearting masonry and served as forms for it. Edges of exposed faces of blocks should be rounded to a radius of about 1 in. In Olive Bridge and Kensico dams very lean, porous, concrete blocks, with 16 in. diam. holes were used for vertical drainage wells near the upstream face. Native stone face blocks were used on Gilboa dam.

Raising heights of existing dams is often required. Often the first construction is but one stage, and dimensions are based on the full-height structure. Sweetwater dam has been twice subject to alterations. In adding 11 ft. to the Trap Falls dam, Bridgeport, use was made of 6-ft. buttresses on 18-ft. centers, bonded to the old structure by mortises 2 ft. × 6 ft., and 6 in. deep, broached in the existing masonry.[66]

Bibliography of Construction

"Ashokan, E. R., Vol. 63, 1911, p. 414. Bassano, *E. N.*, Aug. 37, and Sept. 3, 1914. Camarasa, Spain, *E. N. R.*, Aug. 17, 1922, p. 260. Don Pedro, *E. N. R.*, June 1, 1922, p. 896. Elephant Butte, *E. N.*, Sept. 30, and Oct. 7, 1915. Gilboa dam, *E. N. R.*, Oct. 12, 1922. O'Shaughnessy, *E. N. R.*, Mar. 2, and Sept. 21, 1922. Kensico, *E. R.*, Jan 2, Feb. 13, 1915; *E. N.*, Apr. 9, and May 21, 1914; May 21, 1915; Aug. 10, 1916.

Table 48. State Control of Dams
(E. R., Jan. 6, 1912 forms basis)

State	Supervisory official	State	Supervisory official
California	State water commission		
Colorado	State engineer	New Mexico	State engineer
Connecticut	State board of civil engineers	New York	State engineer
Florida	County commissioners	Oklahoma	State engineer
Georgia	Committee of freeholders	Oregon	State engineer
Idaho	State engineer	Pennsylvania	Dept. of forests and waters
Indiana	County commissioners	Rhode Island	Commissioner of dams and reservoirs; appointed every 3 yrs. by governor
Kansas	County court; laws define methods of construction	South Carolina	Committee of freeholders
Maine	State water storage commission	South Dakota	State engineer
Massachusetts	County commissioners	Tennessee	County court
Michigan	County board of supervisors	Texas	County commissioners' court
Montana	Civil engineer commission appointed by county court	Utah	State engineer
Nebraska	State board of irrigation	Vermont	Commission of three engineers appointed for specific cases by justice of supreme court
Nevada	Local board of water commissioners		
New Hampshire	Public service commission		
New Jersey	Board of three engineers, appointed by governor for each case	West Virginia	Public service commission
		Wyoming	State engineer

Typical Masonry Dams.* In Tables 49 and 50, following, dams are first arranged in classes, overflow and non-overflow, and under the two classes, the arrangement is alphabetical. As in previous tables, use of.......... means that the condition does not exist, and use of — — — — — — means lack of knowledge. "Height† above lowest foundation" is exclusive of cut-off trench. Sections of most dams are on pp. 168 to 187 in almost the order of the table. Maximum pressure is on plane normal to resultant, unless noted otherwise. Consult also special books on dams. "The American Civil Engineers' Hand Book" contains a long list.

* For arch dams, see also p. 139 and p. 144.
† See editorial, "How High is a Dam," *E. N. R.*, Nov. 2, 1922, p. 728.

Table 49. Typical Masonry Dams (Overflow)
Dimensions in Ft.

Name	Austin (Colorado river)‡	Betwa	Boonton	Butte City	Catawba river
Locality	Texas	India	New Jersey	Montana	So. Carolina
Year built	1891–92	1888–97	1900–5	1893–95	1903–04
Height above stream-bed	64	------	60	-----	34
Height above lowest foundation.	68	61.5	95	101	55
Length on top (overflow section).	1091	3296	300	350	585
Minimum thickness at or near top.	Rounded	15	Rounded	20	Rounded
Maximum thickness at or near bottom.°	66	61.5	65	83	33
Radius of upstream face (if curved in plan).	....	Follows reef	....	350	....
Flood depth for which designed.	------	16.4	5	-----	20
Character of foundation	Limestone	Rock ledge	Shale and sandstone	------	Granite
Kind of masonry	Rubble	Granite rubble	Cyclopean syenite	Cyclopean	Cyclopean rubble
Maximum pressure on heel (tons per sq. ft.).	------	------	------	------	------
Maximum pressure on toe (tons per sq. ft.)	------	------	------	------	------
Remarks	Failed; under 11.07 ft. on crest	Usual profile reversed so that 6″ overflow would clear face Has had 16.4 ft. head on crest	------		
References	E. N., July 11, 1891	H. M. Wilson 12th An. Report, U. S. G. S., 1891	Wegmann* E. R., Aug. 8, 1903	E. N., Sept. 5, 1895; Dec. 15, 1892	E. R., July 23, 1904

Table 49. Typical Masonry Dams (Overflow).—(*Continued*)
Dimensions in Ft.

Name	Connecticut river	Dunnings (Scranton)	Escanaba	Folsom	Great Northern Power Co.
Locality	Vernon, Vt.	Pennsylvania	Michigan	California	Minnesota
Year built	1907 10	1887–88	1910	1886–91	1907
Height above stream-bed	39	56	26	70	38
Height above lowest foundation.	70	67±	26	98	40
Length on top (overflow section).	600	152.6	461	180	365
Minimum thickness at or near top.	Rounded	10	Rounded	24	Rounded
Maximum thickness at or near bottom.	33 (*a*)	56	23.3	87	42
Radius of upstream face (if curved in plan).	....	....	....	....	....
Flood depth for which designed.	Sluices in dam	5	2	6	6
Character of foundation	Rock	Rock and clay	Limestone	Bed rock	Rock
Kind of masonry	Cyclopean concrete	Rubble masonry	Cyclopean concrete	Granite ashlar	Concrete
Maximum pressure on heel (tons per sq. ft.).	------	------	------	------	------
Maximum pressure on toe (tons per sq. ft.).	------	------	------	------	------
Remarks	Flashboard 4 ft. high (*a*)28′ below top	....	1-ft. flashboards. 8″ pipes at 10′ intervals drain toe.	Movable shutter raises level 5 ft.	....
References	E. R., Mar. 27, 1909	Trans. Am. Soc. C. E., Vol. 32, 1894	E. R., May 15, 1909	† Schuyler, p. 264	E. R., Sept. 7, 1907

* Wegmann, "The Design and Construction of Dams." ‡ Replaced in 1914. See *E. R.*, June 3, 1915, showing later design. Also D. W. Meade's *Report*, 1917.
† Schuyler, "Reservoirs for Irrigation, Water-Power, and Domestic Water Supply, 1908.
° "Maximum thickness at or near bottom," for overflow dams, includes masonry apron, if any, built as part of the dam section.

Table 49. Typical Masonry Dams (Overflow).—(*Continued*)
Dimensions in Ft.

Name	Hale's Bar	Holyoke (New)	Keokuk (Miss. river)	Lynchburg (Pedlar river)	McCallsFerry (Susquehanna river)
Locality	Tennessee	Massachusetts	Iowa & Ill.	Virginia	Pennsylvania
Year built	1910–13	1897	1911–13	1904–08	1904–10
Height above stream-bed	62	30	32	52	53
Height above lowest foundation.	76	38	37	67.5	60
Length on top (overflow section).	1200	1020	4278	150	2350
Minimum thickness at or near top.	Rounded	Rounded	6	9	Rounded
Maximum thickness at or near bottom.	66	54.2	42	44	65
Radius of upstream face (if curved in plan).	....	....	....	....	....
Flood depth for which designed.	------	4	11	6	17.5
Character of foundation	Limestone seamy in spots	------	Limestone	Shale	Hard gneiss
Kind of masonry	Concrete	Rubble	Concrete	Concrete and granite	Cyclopean rubble
Maximum pressure on heel (tons per sq. ft.).	------	------	------	------	4.8
Maximum pressure on toe (tons per sq. ft.).	------	------	------	------	6.2
Remarks	Caisson foundations	....	------	Air pipes prevent vacuum at face.	....
References	E. R., Feb. 15, 1913. E. N., Nov. 13, 1913	E. N., 1897, Vol. 37, p. 292	E.N., Sept. 28, 1911	E. R., May 12, 1906; July 23, 1904	E. R., May 28, 1910

Table 49. Typical Masonry Dams (Overflow).—(*Continued*)
Dimensions in Ft.

Name	Muscoot	Scioto (Columbus)	Spier Falls	Turlock (LaGrange)	Vyrnwy
Locality	New York	Ohio	New York	California	England
Year built	1902	1904–5	1900–05	1891–94	1882–90
Height above stream-bed	35	33	52	127.5	101
Height above lowest foundation.	70	45.5	70	127.5	162
Length on top (overflow section).	200	500	817	309	1172
Minimum thickness at or near top.	5.5	Rounded	Rounded	Rounded	Rounded
Maximum thickness at or near bottom.	31.5	64.3	65	92	120
Radius of upstream face (if curved in plan).	....	800	....	308.0	....
Flood depth for which designed.	------	22	------	16	1.5
Character of foundation	Rock	Limestone	Granite	Rock	Clay, slate, rock
Kind of masonry	Rubble masonry	Concrete	Rubble, cyclopean	Blue trap rubble	Cyclopean rubble
Maximum pressure on heel (tons per sq. ft.)	------	------	------	------	8.7
Maximum pressure on toe (tons per sq. ft.).	------	------	------	------	6.36
Remarks	Downstream face stones carefully selected and bedded to withstand ice	Allowance for future addition 22′ high	....	Has had 15 ft. head on crest	Foundation rock drained to prevent uplift
References	E. R., Dec. 20, 1902	T.A.S.C.E., Vol. 67, 1910	E. N., Jun. 18, 1903	E. N., 1894, Vol. 31, p. 266	Proc. Inst. C. E., Vol. 126, 1895, p. 29

Table 50. Typical Masonry Dams (Not Overflow) Dimensions in Ft.

Name	Ashokan (Olive Bridge)	Assuan	Barker	Barossa	Burrin Juck
Locality	New York	Egypt	Colorado	S. Australia	N.S W.
Year built	1907–11	1898–05	1909	1899–03	1908
Height above stream-bed	150	82(a)	145	95	224
Height above lowest foundation (exclusive of cut-off trench).	220	131	172	113	240
Length on top (omitting spillway)	1000	6400*	625	472	784
Minimum thickness at or near top	26.3	17.8	16	4.5	18
Maximum thickness at or near bottom	190.2	85	124	45	160
Radius of upstream face (if curved in plan)	....	....	....	200	1200
Height of top above full res. level	20	13.1	------	2	12
Character of foundation	Hamilton shale	Granite ledge	Granite, 10 ft. cut-off wall 10 ft. deep	Shale	Granite
Kind of masonry	Cyclopean	Red granite rubble	Cyclopean	Cyclopean	Cyclopean
Maximum pressure on heel (tons per sq. ft.).	14†	6.5†	------	Limit 15	16.8
Maximum pressure on toe (tons per sq. ft.)	8.2†	4.5†	------		15.5
Allowed ice thrust, tons per lin. ft. at full res. level.	23.5	None	------	------	------
Allowed for upward water pressure	Full head at heel and 0 at toe, on ⅔ area of joint	None	------	------	------
Remarks	Continuous drains.	180 sluices each 6.56 ft. wide	Expansion joints 48 ft. apart.	Rail reinf. at top	Factor of safety, 2.16
References	....	E. N., Sept. 30, 1909; Cassier's, Feb., 1903	E. R., Oct. 2, 1909	E. R., Sept. 2, 1905; E. N., Apr. 7, 1904	Engineer, Sept. 23, 1910

* Including both pierced and solid dam. (a) Increased 16 ft. in 1911 to 98 ft. † Vertical pressures.

Table 50. Typical Masonry Dams (Not Overflow).—(*Continued*) Dimensions in Ft.

Name	Bear Valley (old)	Beetaloo	Boonton	Boyd's Corners	Burraga
Locality	California	S. Australia	New Jersey	New York	Australia
Year built	1884	1888–90	1900–05	1866–72	1893
Height above stream bed	------	110	105	58.0	39
Height above lowest foundation (exclusive of cut-off trench).	64	118	114	78.0	41.0
Length on top (omitting spillway)	300	367	1850	670	425.6
Minimum thickness at or near top	3.17	14	17	8.6	2.0
Maximum thickness at or near bottom	8.4 at 48 below top 20 max.	110	77	57	25.3
Radius of upstream face (if curved in plan)	335	1428	....	....	539.8
Height of top above full res. level	4	5	5.0	3	2.5
Character of foundation	Granite	Shale and quartzite	Shale and sandstone	Rock	Slate
Kind of masonry	Granite rubble	Concrete	Cyclopean of syenite	Concrete hearting	Cyclopean
Maximum pressure on heel (tons per sq. ft.)	Limit, 40	7.0	9	------	Limit, 10
Maximum pressure on toe (tons per sq. ft.)		4.6	8	------	
Allowed ice thrust, tons per lin. ft. at full res. level.	------	------	None	------	------
Allowed for upward water pressure	------	------	None	------	------
Remarks:	Max. arch stress = 59 tons per sq. ft.	Temperature cracks were grouted	....	Cut-stone facing	Rails at top for expansion
References	E. N., June 23, 1888	Proc. Inst. C. E., 1892, Vol. 113, p. 151	E. R., 1903, Vol. 48, p. 153	Wegmann	Proc. Inst. C. E., Vol. 152, 1903

Table 50. Typical Masonry Dams (Not Overflow).—(*Continued*)
Dimensions in Ft.

Name	Burrator	Cataract	Chartrain	Chemnitz (Einsiedel)	Cross River
Locality	England	Australia	France	Germany	New York
Year built	1893–96	1902–08	1888–92	1894	1905–07
Height above stream-bed	89	157	163	65	155
Height above lowest foundation (exclusive of cut-off trench).	145	192	180	92	170
Length on top (omitting spillway)	361	811	————	590	772
Minimum thickness at or near top	18	16.5	13.12	13	23
Maximum thickness at or near bottom	63†	158	159.9	65.5	116
Radius of upstream face (if curved in plan)	....	....	1319	1316.5	....
Height of top above full res. level	12±	6.6	1.5	1.64	10
Character of foundation	Granite bed	Sandstone	Rock	Slate	Gneiss
Kind of masonry	Granite blocks in concrete	Cyclopean	Granite rubble	Rubble	Cyclopean
Maximum pressure on heel (tons per sq. ft.)	————	*	Limited to 11.3	————	10.0
Maximum pressure on toe (tons per sq. ft.)	————	*		————	12.5
Allowed ice thrust, tons per lin. ft. at full res. level.	————	None	————	————	12
Allowed for upward water pressure	————	None	————	————	See Ashokan
Remarks	....	Drains		....	....
References	Proc. Inst. C. E., Vol. 146, 1901	E. N., Dec. 6, 1906; June 23, 1910; E. R., June 6, 1908	Wegmann	E. R., July 28, 1894	E. R., Sept. 14, 1907; June 16, 1906

* Maximum allowable pressure on heel and toe taken small (9.5 tons) on account of moisture-laden sandstone used.
† 77 ft. below water level.

Table 50. Typical Masonry Dams (Not Overflow).—(*Continued*)
Dimensions in Ft.

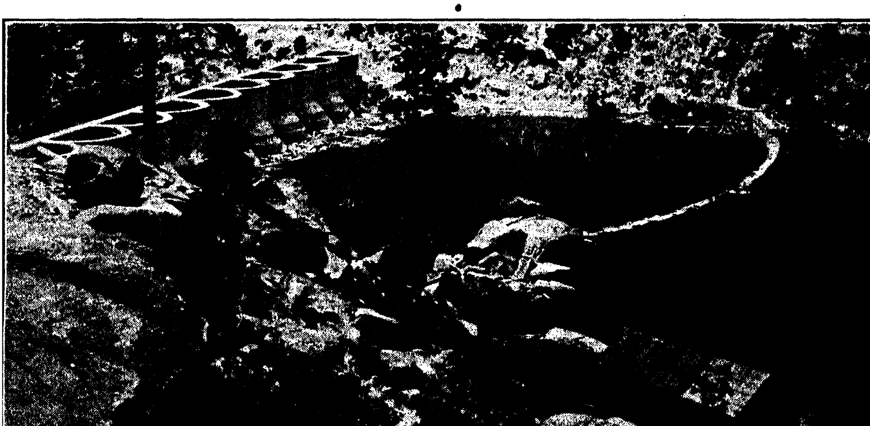
Name	Croton Falls	Furens	Gileppe	Granite Springs	Hemet
Locality	New York	France	Belgium	Wyoming	California
Year built	1906–11	1862–66	1869–75	1903–04	1890–95
Height above stream-bed	113	170.6	151	86±	122.5
Height above lowest foundation (exclusive of cut-off trench).	167	183.7	154	96	135.5
Length on top (omitting spillway)	1095	330	771	410	260
Minimum thickness at or near top	23	9.9	49.2	10	10
Maximum thickness at or near bottom	127.7	161	215.9	56	100
Radius of upstream face (if curved in plan)	....	841.4	1665	300	225.4
Height of top above full res. level	12	6.56	6.6	3	1.0
Character of foundation	Granite	Mica schist	Rock	Gabbro	Bedrock
Kind of masonry	Cyclopean	Rubble	Rubble	Uncoursed rubble	Granite rubble
Maximum pressure on heel (tons per sq. ft.).	10.0	Limited to 6.6	————	————	————
Maximum pressure on toe (tons per sq. ft.).	12.5		6.14	————	————
Allowed ice thrust, tons per lin. ft. at full res. level.	15	————	————	————	————
Allowed for upward water pressure	See Ashokan	————	Sp. gr. of stone reduced from 2.3 to 1.3	————	————
Remarks	....	....	....	....	Designed for ultimate height of 160 ft.
References	Proc. Mun. Eng., 1910; E. R., Mar. 28, 1908	Wegmann	Wegmann E. N., Dec. 25, 1886	E. R., 1905, June 24, p. 698; E. N., 1905, June 29, p. 671	18th An. Rept., U. S. G. S., 1897

For Crystal Springs dam, see San Mateo, p. 166.
For Croton or Cornell dam, see New Croton, p. 165

Table 50. Typical Masonry Dams (Not Overflow).—(*Continued*)

Name	Hijar	Kensico	Lake Cheesman	New Croton	Pathfinder
Locality	Spain	New York	Colorado	New York	Wyoming
Year built	1880	1910-16	1900-04	1892-1907	1906-09
Height above stream-bed	------	170	214	163	204
Height above lowest foundation (exclusive of cut-off trench).	141.1	300	236	297	218
Length on top (omitting spillway)	236.2	1630ϕ	710	1168	425
Minimum thickness at or near top	16.4	27.75	18	18	10
Maximum thickness at or near bottom	147	227.7	176	206	94
Radius of upstream face (if curved in plan)	218	....	400	....	157
Height of top above full res. level	4.9	15	3	10	10
Character of foundation	Rock	Schist, limestone and gneiss	Granite	Seamy rock	Red granite
Kind of masonry	------	Cyclopean	Granite rubble	Rubble	Cyclopean
Maximum pressure on heel (tons per sq. ft.)	6.0	15*	(a) (b) 12.8 13.4*	†	Designed as horizontal arch including temp. stresses. Max. = 13 tons
Maximum pressure on toe (tons per sq. ft.)	5.1	13*	17.7 13.9	†	
Allowed ice thrust, tons per lin. ft. at full res. level.	------	23.5	None	None	------
Allowed for upward water pressure	------	Full head at heel and 0 at toe, applied on ⅔ area of joint	None	None	------
Remarks	....	Continuous drains. Stones weighed 1 to 16 tons	(a) Computed as gravity section; (b) arch and cantilever	Spillway 1000 ft. long	....
References	Wegmann	ϕExclusive of pavilions	T. A. S. C. E., Vol 53, 1904 p. 89	E. R., Aug. 16, 1902; E. N., Oct. 4, 1906	E N., Oct. 29, 1908, E R., Nov. 7, 1908

* Vertical. † Profile is based on Quaker Bridge dam, which had maximum heel pressure of 16.6 tons; maximum toe pressure of 15.4 tons (Wegmann.)



Old and new Bear Valley dams.
(*Eng. News, Oct.* 28, 1915.)

Table 50. Typical Masonry Dams (Not Overflow).—(*Continued*)
Dimensions in Ft.

Name	Periyar	Roosevelt	San Mateo*	Shoshone	Sodom
Locality	India	Arizona	California	Wyoming	New York
Year built	1888–96	1905–11	1887–88	1905–10	1888–93
Height above stream-bed	155	262	146	243	78
Height above lowest foundation (exclusive of cut-off trench).	178	284	162	328	98
Length on top (omitting spillway)	1231	1080	680	175	500
Minimum thickness at or near top.	12	16	25	10	12
Maximum thickness at or near bottom.	136	170	176	108	53
Radius of upstream face (if curved in plan).	....	418	645	155	....
Height of top above full res. level.	5	20	8	10	10
Character of foundation	Bedrock (syenite)	Bedrock, sandstone	Fissured sandstone	Solid granite	Rock
Kind of masonry	Concrete	Cyclopean	Concrete blocks	Concrete	Coursed rubble
Maximum pressure on heel (tons per sq. ft.).	>9	22†	———	Designed as horizontal arch including temp. stresses	———
Maximum pressure on toe (tons per sq. ft.).	>9	23†	———		———
Allowed ice thrust, tons per lin. ft. at full res. level.	None	———	———	———	None
Allowed for upward water pressure.	———	———	Whole head	———	None
Remarks	....	....	See p. 125	....	....
References	12th An. Rept., U. S. G. S., 1891	E. N., 1905, Vol. 53, p. 34; Sept. 10, 1908, p. 265	U. S. G. S., 18th Report, Part IV, 1897	E. N., Dec. 9, 1909; E. R., July 23, 1910	T. A. S. C. E., Mar., 1893

* Also known as Crystal Springs dam. † "Construction of Masonry Dams," C. W. Smith, 1915.

Table 50. Typical Masonry Dams (Not Overflow).—(*Continued*)
Dimensions in Ft.

Name	Spier Falls	Sweetwater	Sweetwater (Extension)	Thirlmere	Titicus
Locality	New York	California	California	England	New York
Year built	1900–05	1887–88	1910–11*	1886–93	1890–95
Height above stream-bed	90	75	110	58.5	109
Height above lowest foundation (exclusive of cut-off trench).	154	94	129	112	135
Length on top (omitting spillway).	552	380	380	———	534
Minimum thickness at or near top.	17	12	12	18.5	20.7
Maximum thickness at or near bottom.	113	46	76	51.7	81.4
Radius of upstream face (if curved in plan).	....	222	222	118.5	....
Height of top above full res. level.	10	5.5	5	6.0	5
Character of foundation	Granite	Porphyry	Porphyry	Ledge rock	Rock
Kind of masonry	Rubble, cyclopean granite	Rubble	Cyclopean	Cyclopean concrete	Rubble, rough coursed
Maximum pressure on heel (tons per sq. ft.).	6.6	5.8	8.14(*a*)	———	———
Maximum pressure on toe (tons per sq. ft.).	6.6	15.8	9.45(*a*)	———	———
Allowed ice thrust, tons per lin. ft. at full res. level.	———	None	None	———	———
Allowed for upward water pressure.	———	———	———	———	———
Remarks	Silt in water for 60 ft. at mid-depth, sp. gr. = 1.6	In 1895, dam was over-topped with 22 in. water for 40 hrs. without injury	Addition bonded to face of old dam (*a*) Vertical	Plan, reverse curve, to follow ledge	....
References	E. N., 1903, Vol. 49, p. 553	T. A. S. C. E., Vol. 19, 1888, p. 201	E. N., Mar. 30, 1911, p. 371	Proc. Inst. C. E., Vol. 126, 1895, p. 5	E. R., 1895, Vol. 32, p. 58

* Reconstruction of top in 1916. See *E. N. R.*, May 15, 1919, p. 948.

Table 50. Typical Masonry Dams (Not Overflow).—(*Concluded*)
Dimensions in Ft.

Name	Upper Otay†	Urft	Villar	Wachusett	Wigwam (Waterbury)	Zola*
Locality	California	Germany	Spain	Massachusetts	Connecticut	France
Year built	1900	1901–04	1870–78	1900–06	1893–94	1843
Height above stream-bed	———	175	162	146	77	119.7
Height above lowest foundation (exclusive of cut-off trench).	84	190	170.33	207	91	123.2
Length on top (omitting spillway).	350	741	349	850	600	205
Minimum thickness at or near top.	4	18	14.75	22.5	12	19
Maximum thickness at or near bottom.	14	165.5	154.6	187	62.08	41.8
Radius of upstream face (if curved in plan).	361	656	447	....	600	158
Height of top above full res. level.	———	3	8.25	20	7	———
Character of foundation	Porphyry	Granite schist	Rock	Granite and schist	Gneiss and sandstone	———
Kind of masonry	Masonry reinforced by plates and cables	Trap masonry	Rubble	Granite rubble	Rubble masonry	Rubble
Maximum pressure on heel (tons per sq. ft.).	———	———	———	12.0*	———	8.12, 62′ above base
Maximum pressure on toe (tons per sq. ft.).	———	———	9.60	10.7*	———	
Allowed ice thrust, tons per lin. ft. at full res. level.	———	———	———	23.5	———	———
Allowed for upward water pressure.	———	None	———	Full head at heel, 0 at toe applied on $\frac{2}{3}$ area of joint	———	———
Remarks	....	Continuous drains	....	....	Designed for 90 ft. height	Under full head pressure line falls 11.5 ft. outside of base
References	.... Wegmann	E. N., July 16, 1903; Engineering, Nov., 1907	Proc. Inst. C. E., Vol. 71, p. 379, 1883	E. R., Sept. 8, 1900; E. R., Oct. 6, 1906	E. N., May 7, 1903	Wegmann

* Vertical. † 3 ft. of water passed over crest, Jan., 1916, without damage. *E. R.*, Feb. 12 and 19, 1916.

Arrowrock dam, of U. S. Reclamation Service on Boisé River, Idaho, is one of the highest. Built 1912–1915. Maximum height above foundation, 351 ft.; above river level, 260 ft. Length on top, 1050 ft. Minimum thickness, 15.5 ft.; maximum, 238 ft. Radius of center line of top, 661.74 ft. Parapets on each side of roadway extend 4 ft. above top of dam. There is a waste weir 400 ft. long, separate from the dam, with crest about 5 ft. below top of dam; but the dam is designed to permit a flow 2 or 3 ft. deep over its top in great floods. Cross-section is of gravity type, with vertical upstream face, the portion below 100 ft. being offset 3 ft. upstream. Foundation is hard granite. The dam is of concrete cast against wooden forms; proportions: 1 sand cement, 2½ sand, 5½ gravel, and 2¾ parts cobbles, with a richer mixture for 10 ft. from each face. Sand cement was made of 55 per cent. Portland cement and 45 per cent. pulverized granite, reground to pass 90 per cent. through a sieve having 200 meshes per in. There is a 20-ft. inspection gallery, 25 ft. from the upstream face, with its bottom 235 ft. below top of dam.

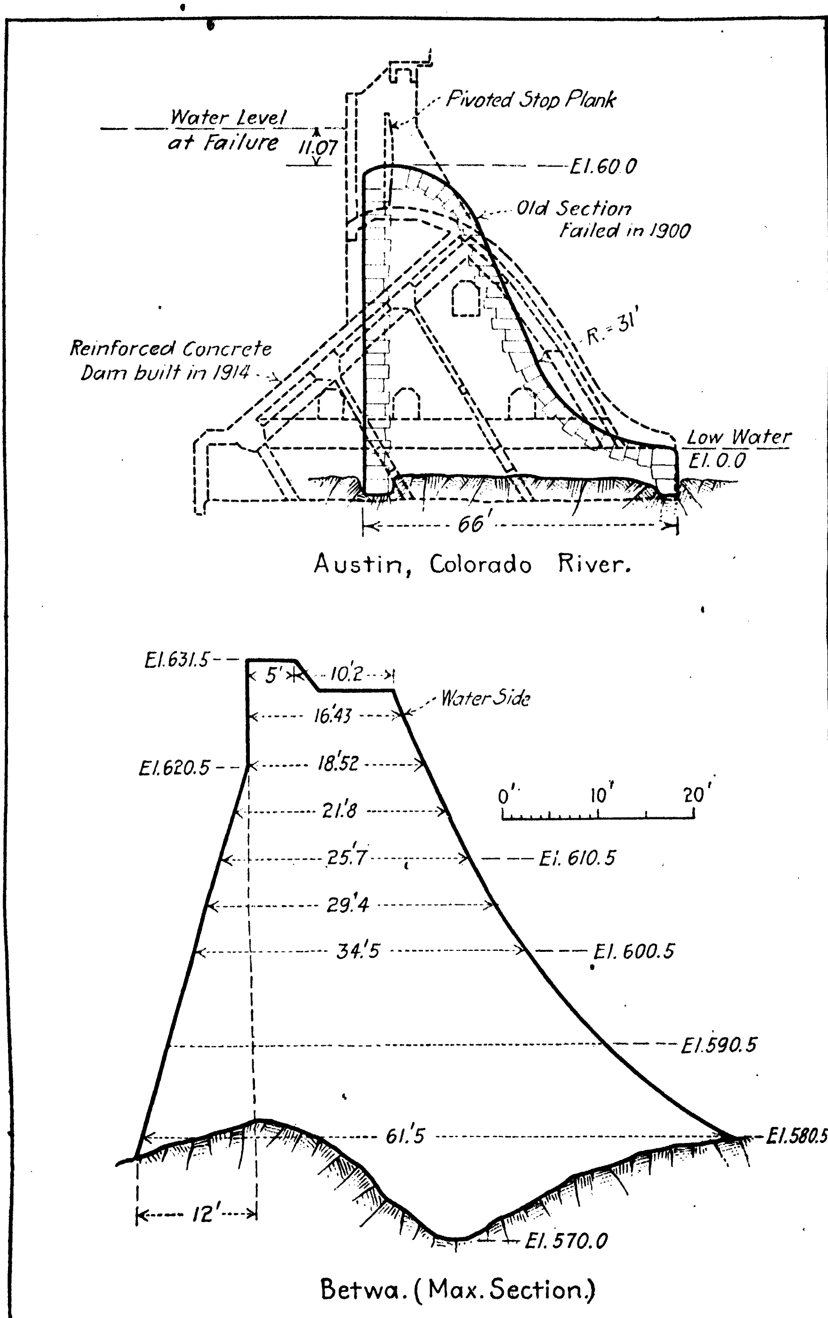


FIG. 59.—Overflow dams.

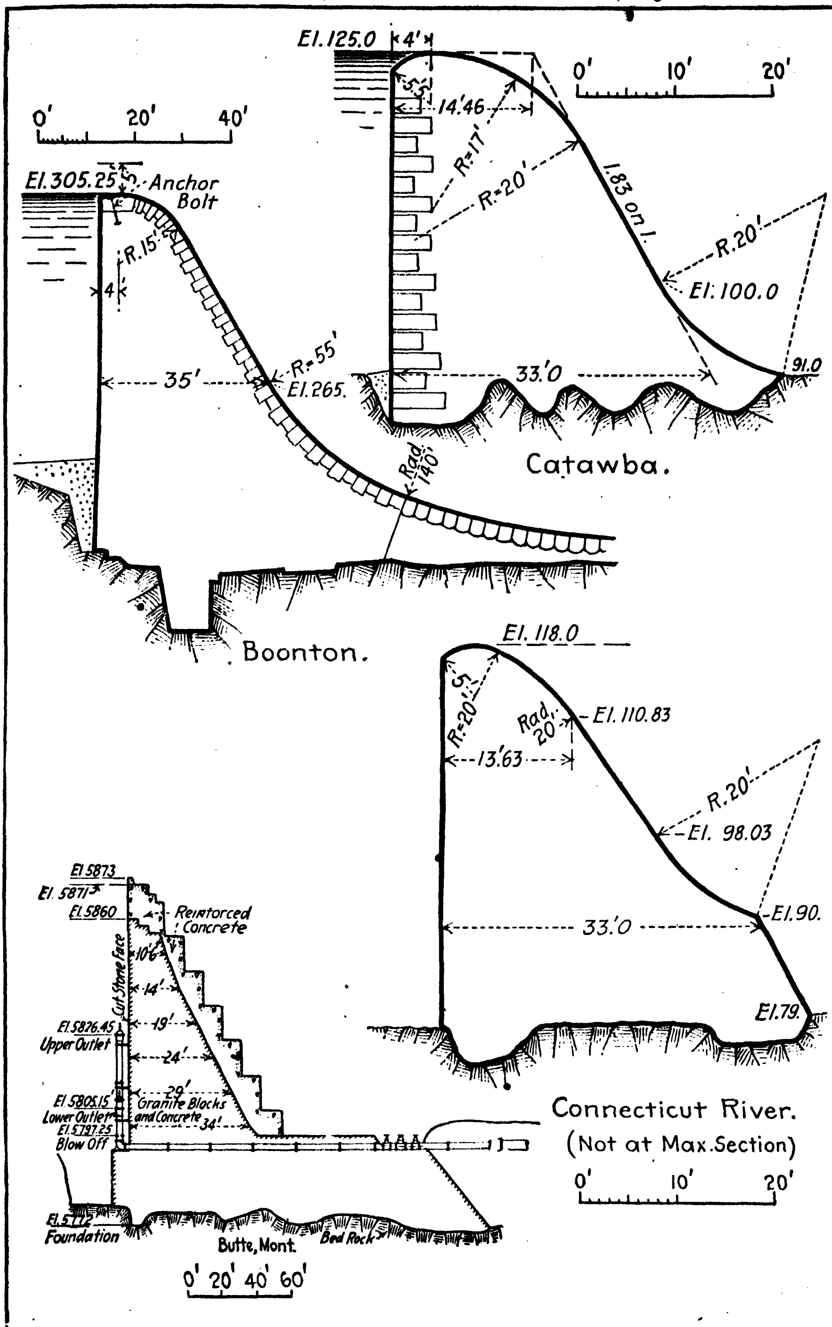


FIG. 60.—Overflow dams.

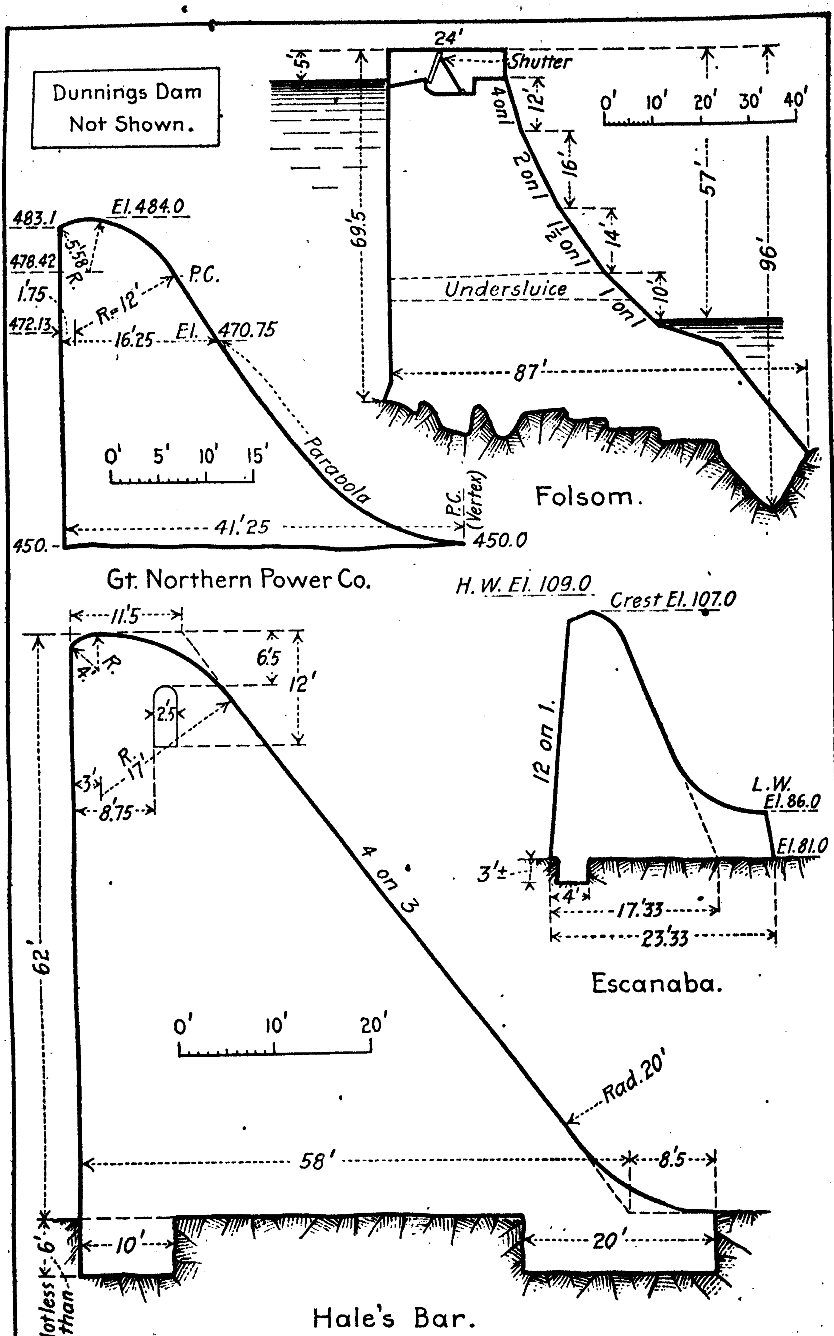


FIG. 61.—Overflow dams.

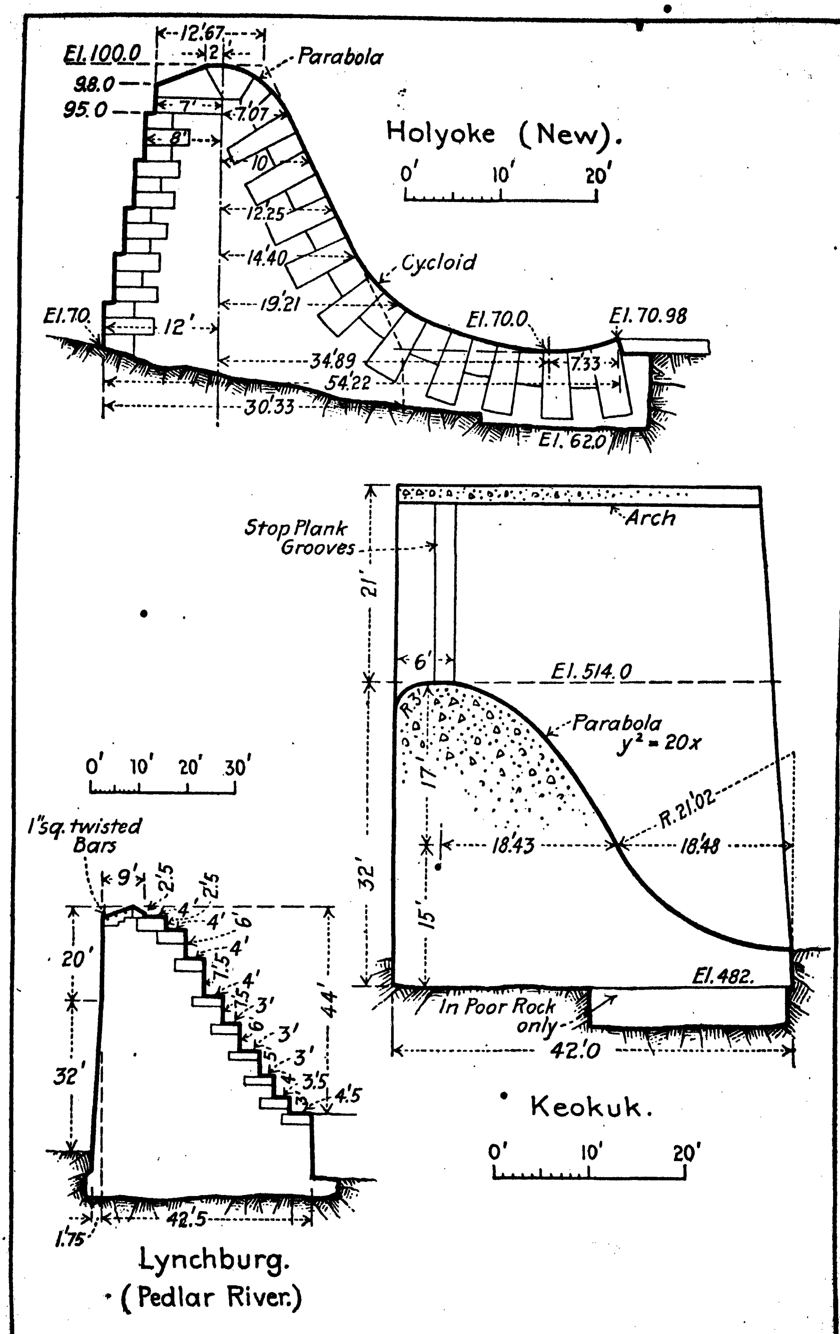


FIG. 62.—Overflow dams.

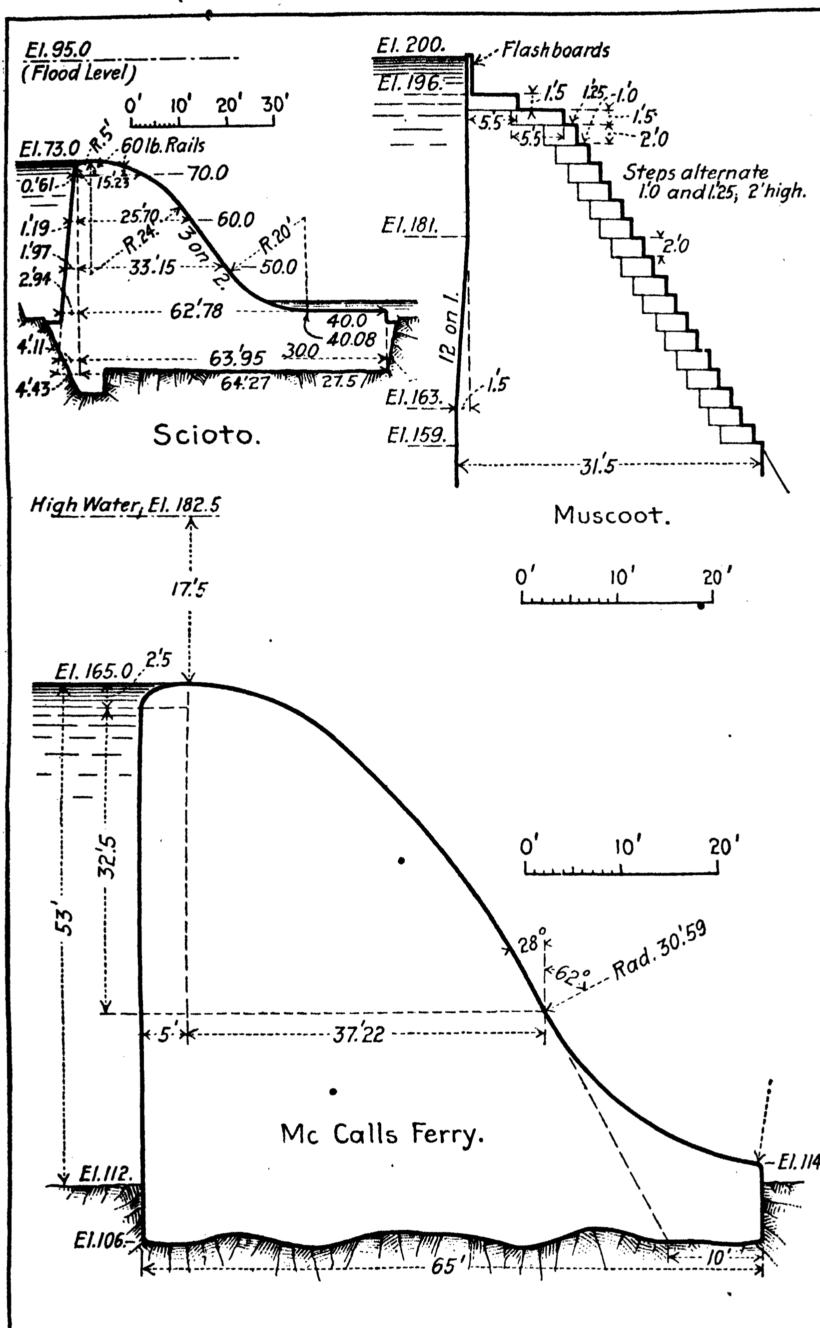


FIG. 63.—Overflow dams.

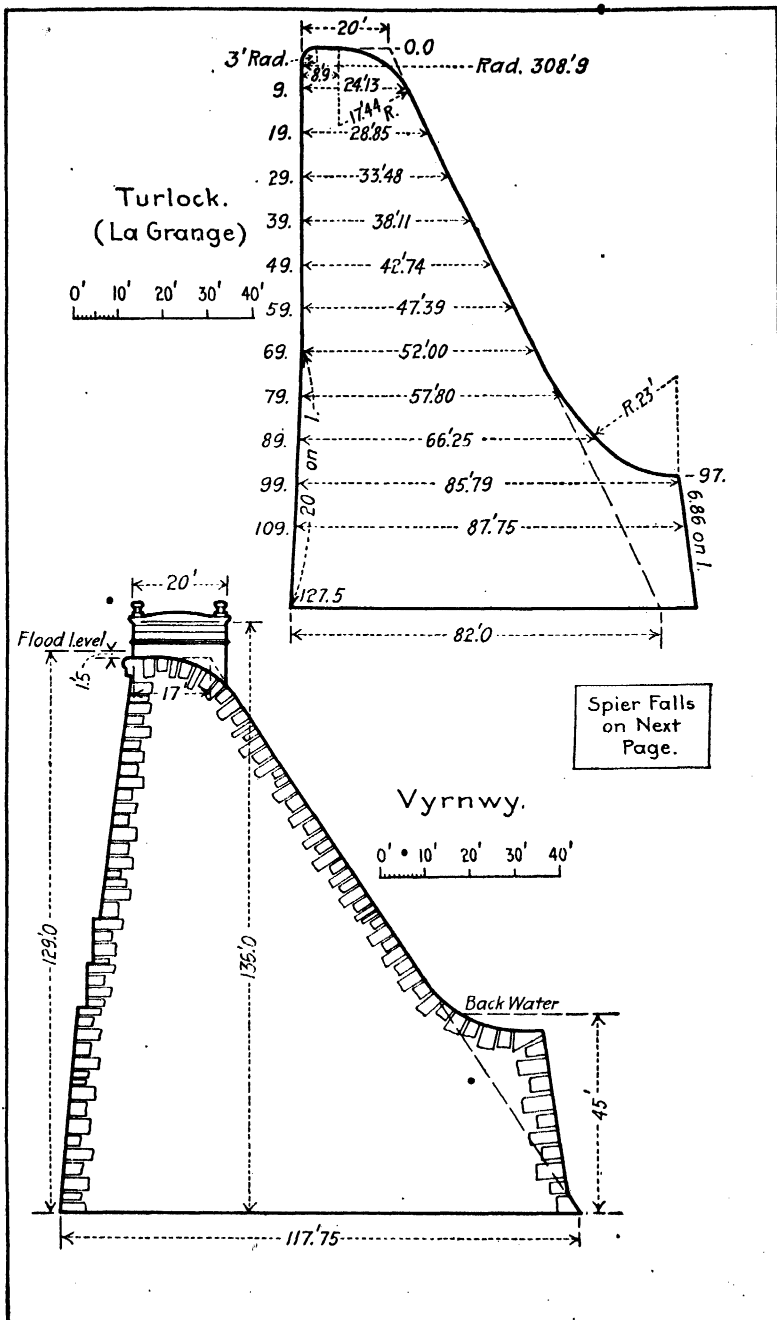


Fig. 64.—Overflow dams.

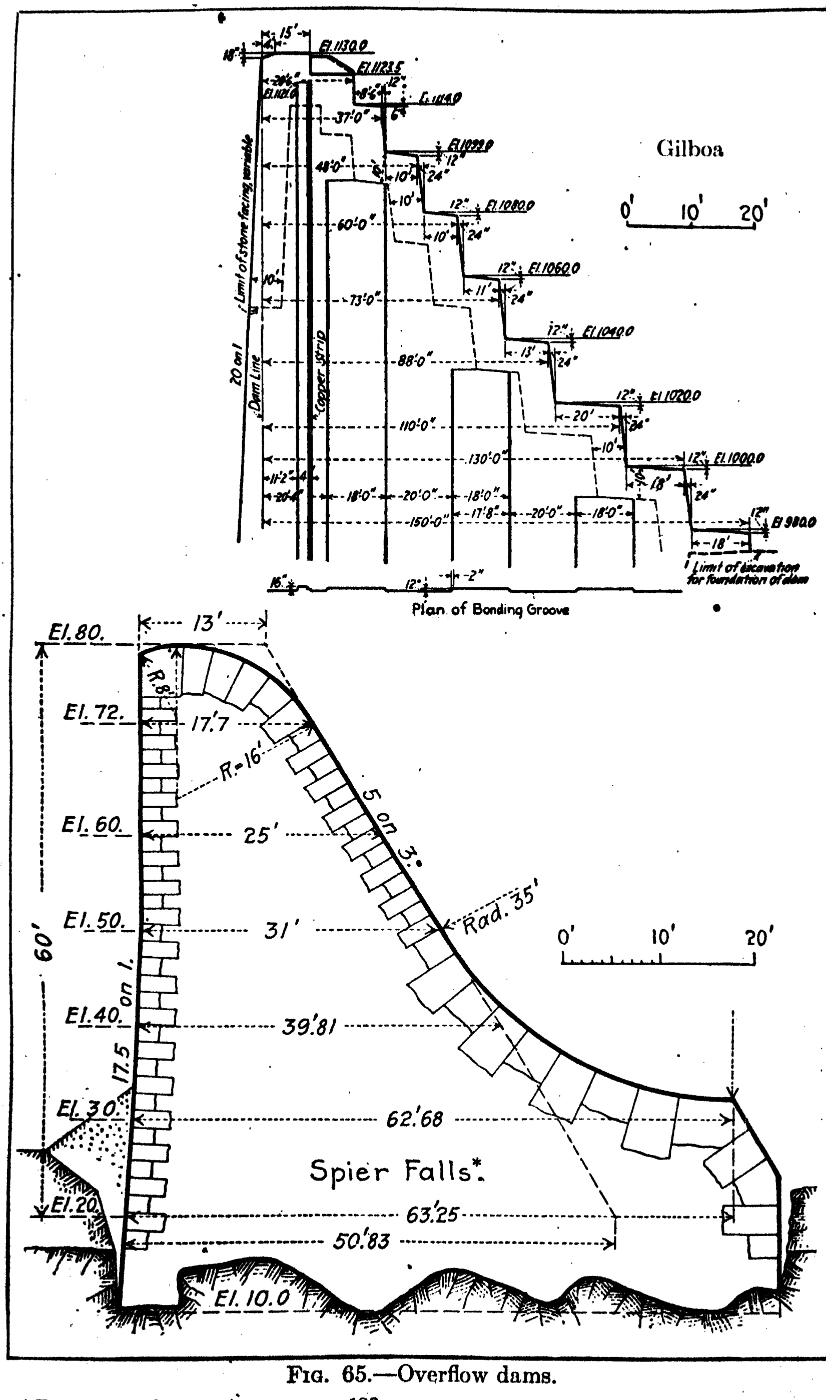


FIG. 65.—Overflow dams.

* For non-overflow portion, see page 183.

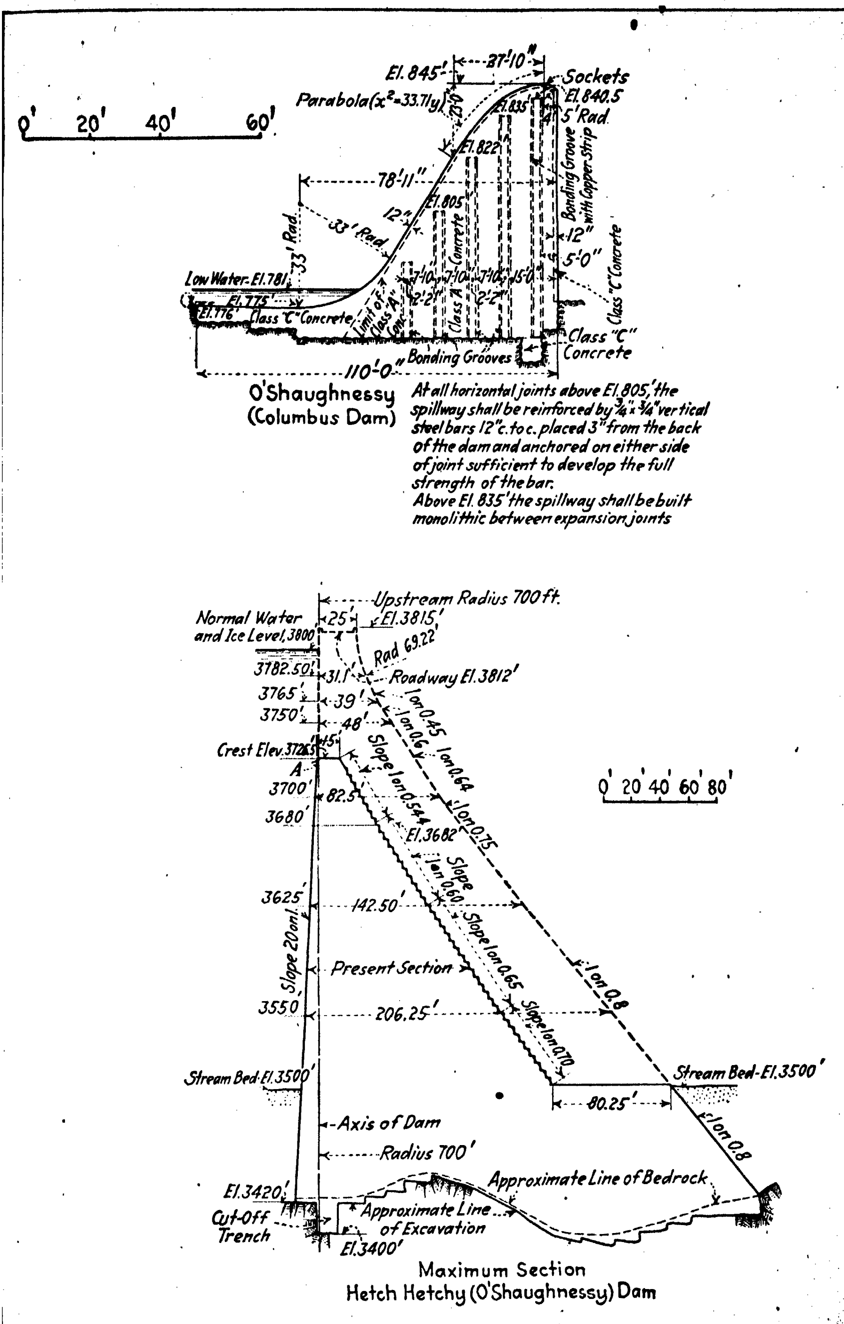


FIG. 66.—O'Shaughnessy dams.

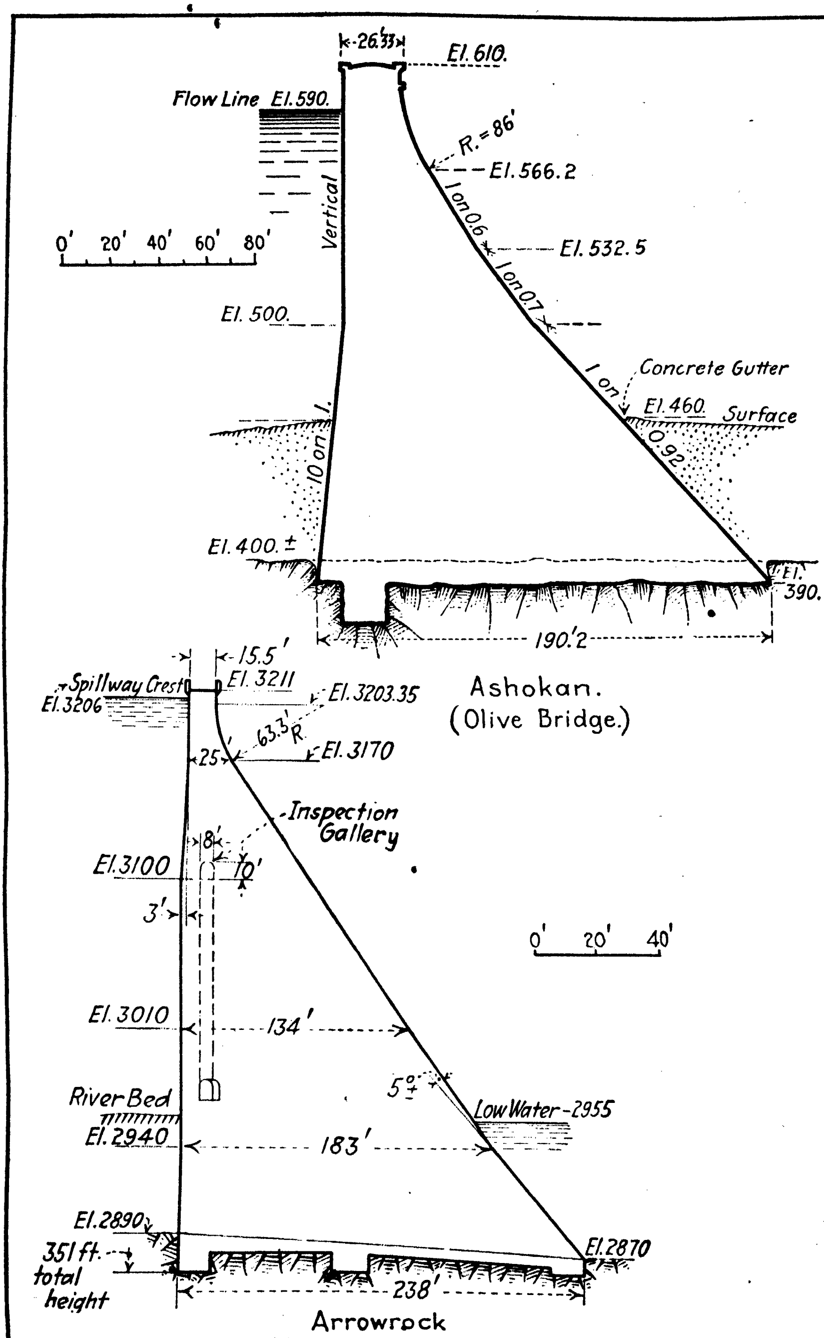


FIG. 67.—Non-overflow dams.

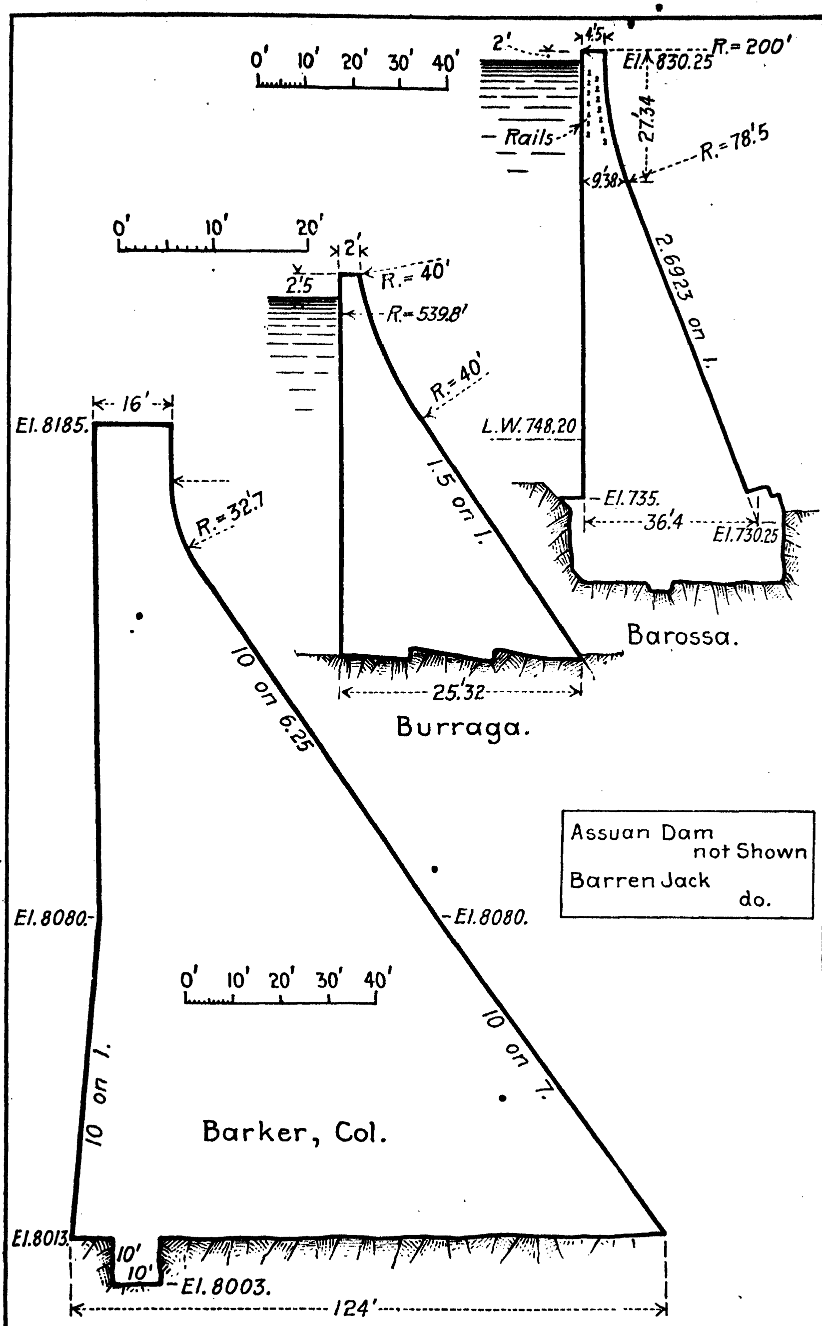


FIG. 68.—Non-overflow dams.

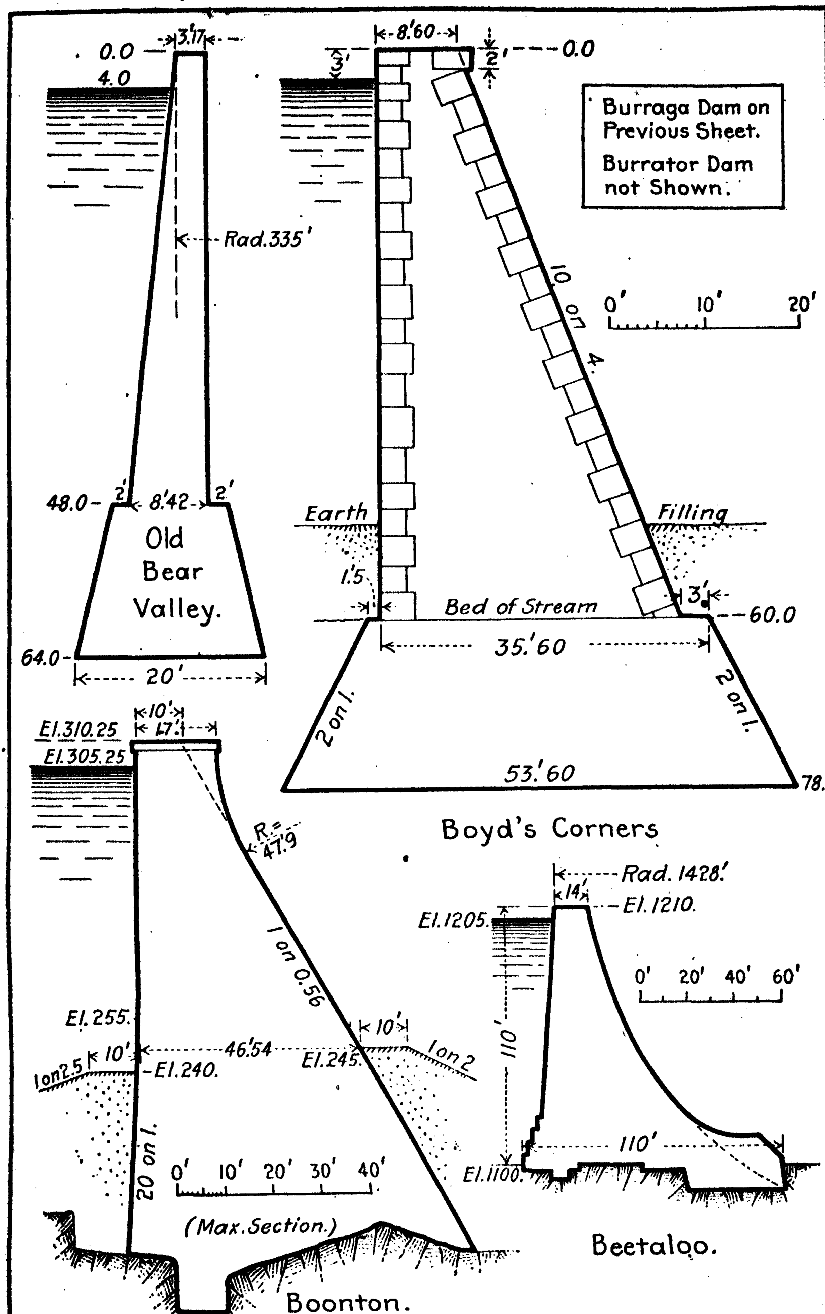


FIG. 69.—Non-overflow dams.

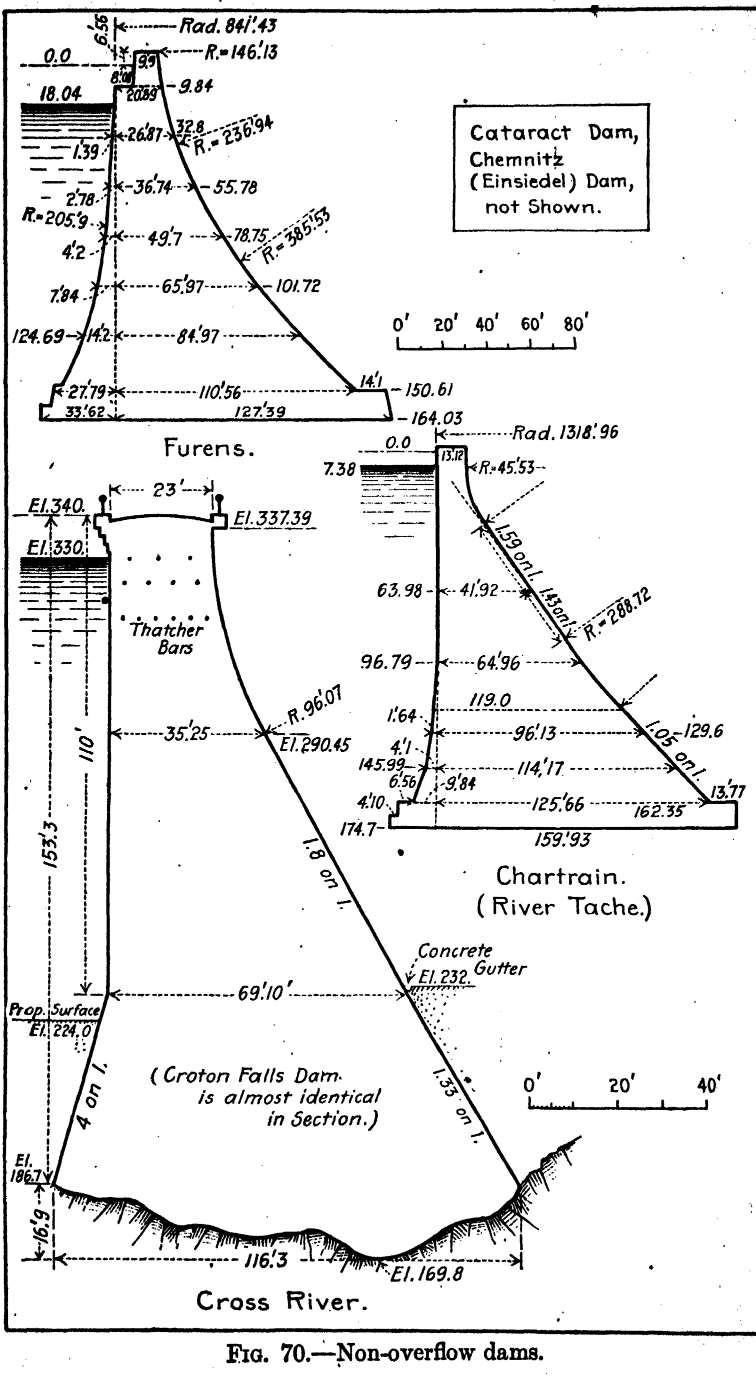
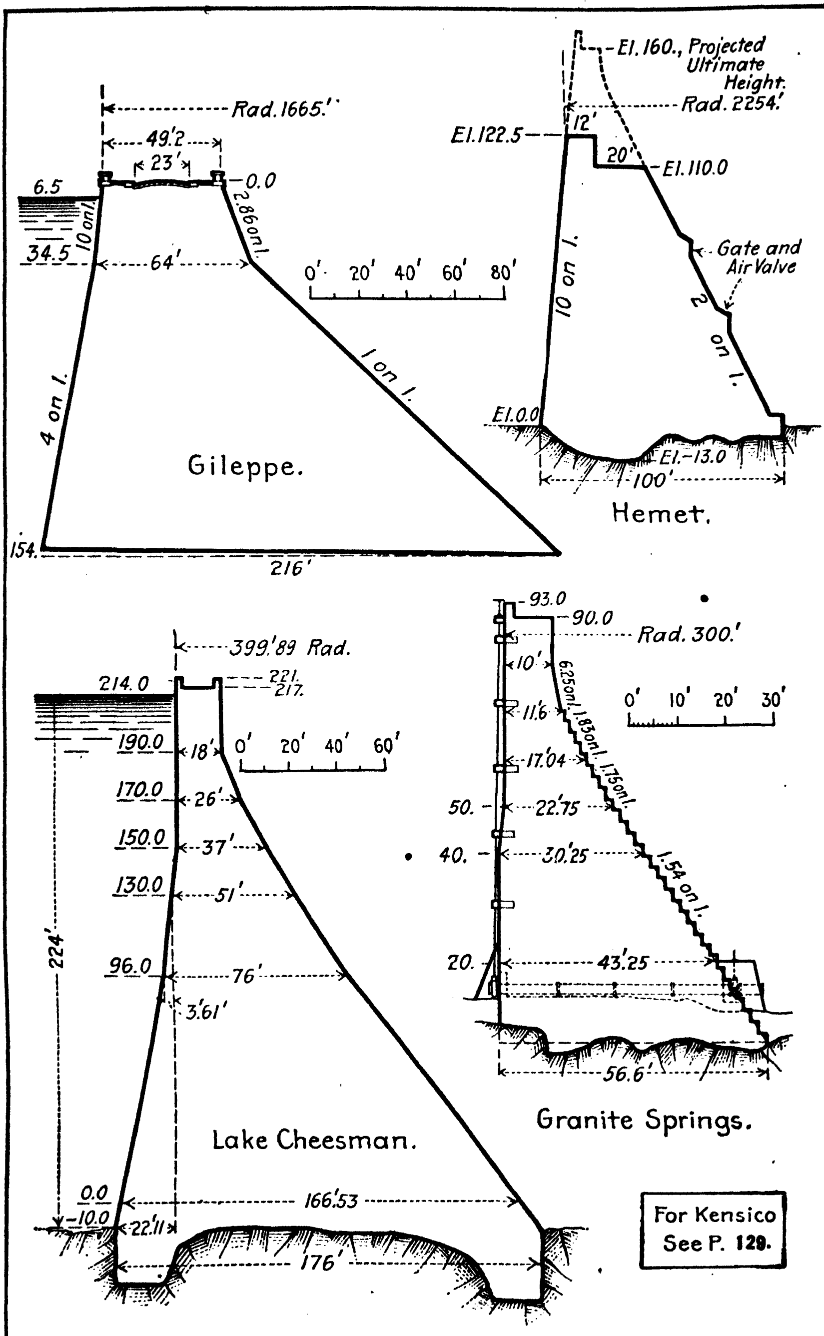


FIG. 70.—Non-overflow dams.



For Kensico See P. 129.

FIG. 71.—Non-overflow dams.

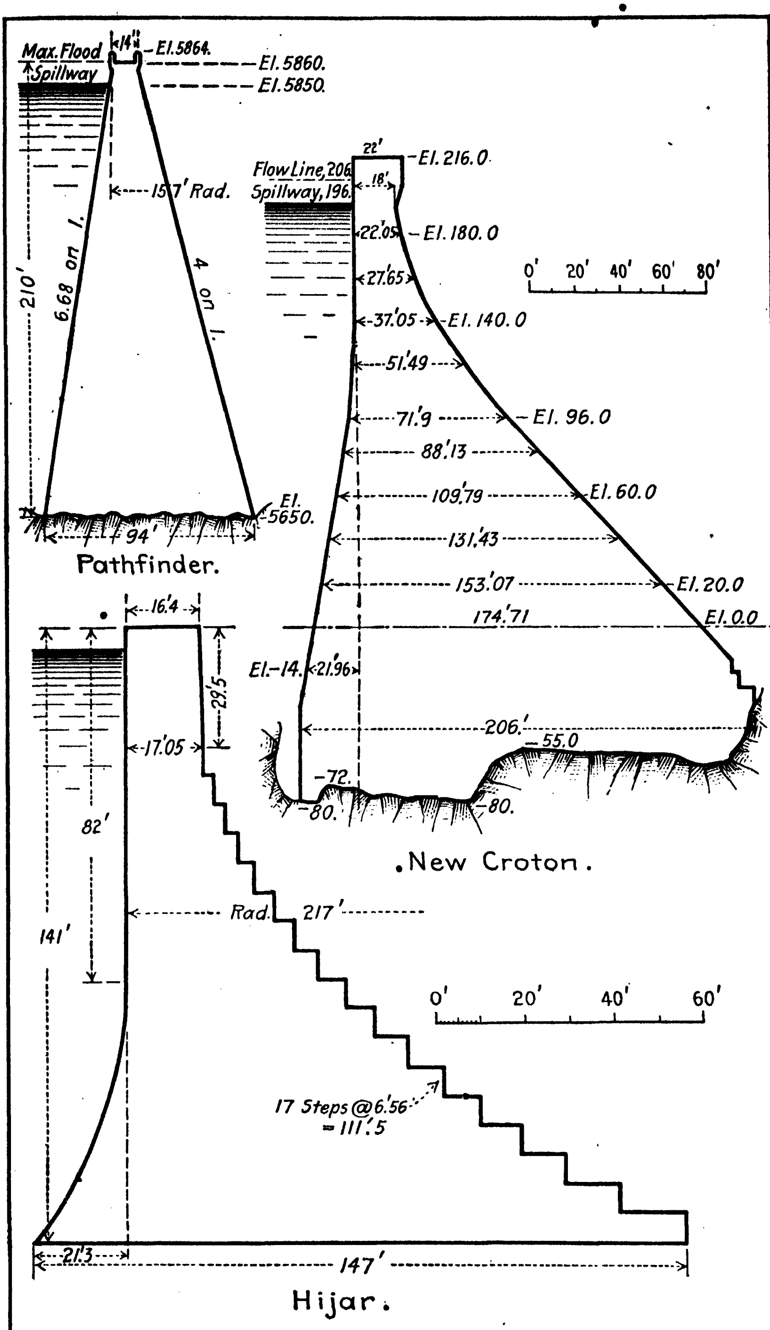


Fig. 72.—Non-overflow dams.

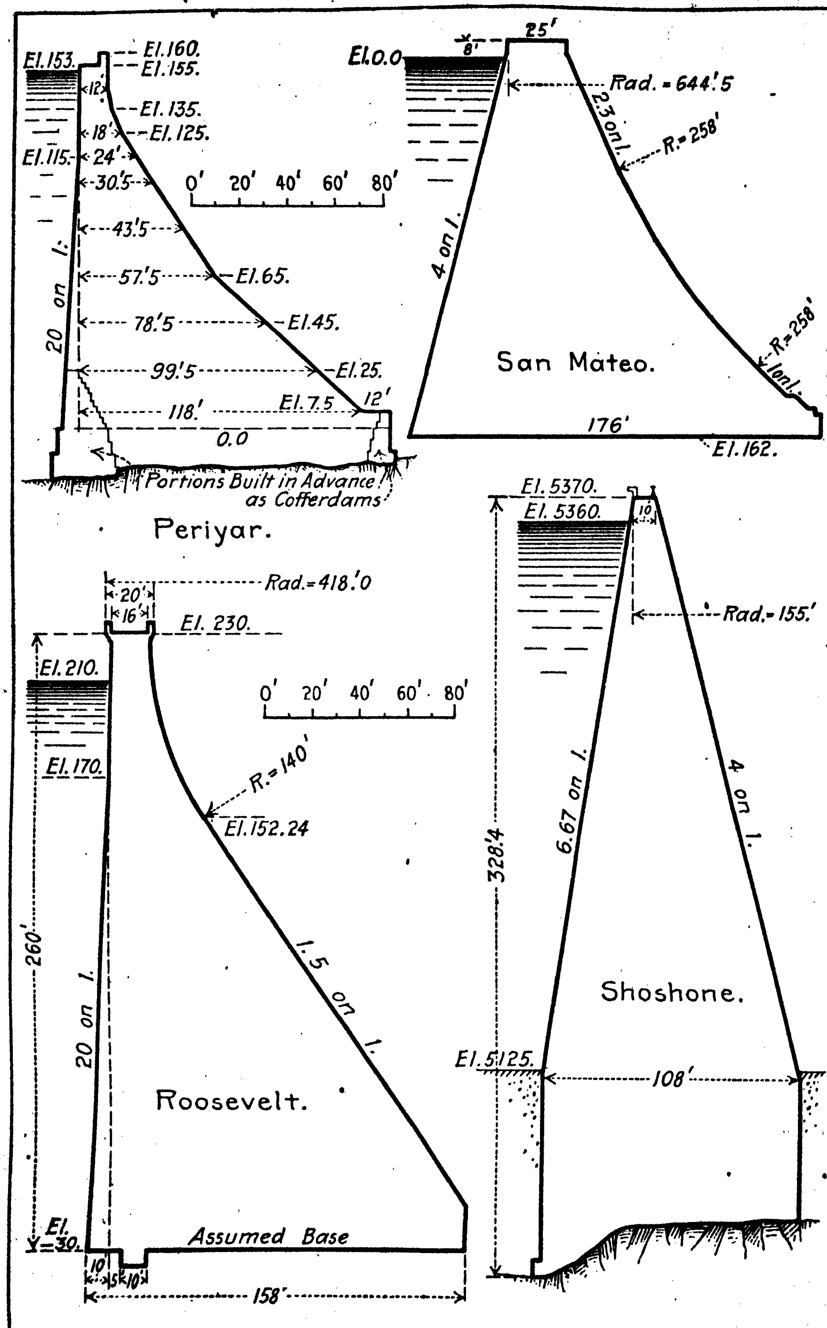


FIG. 73.—Non-overflow dams.

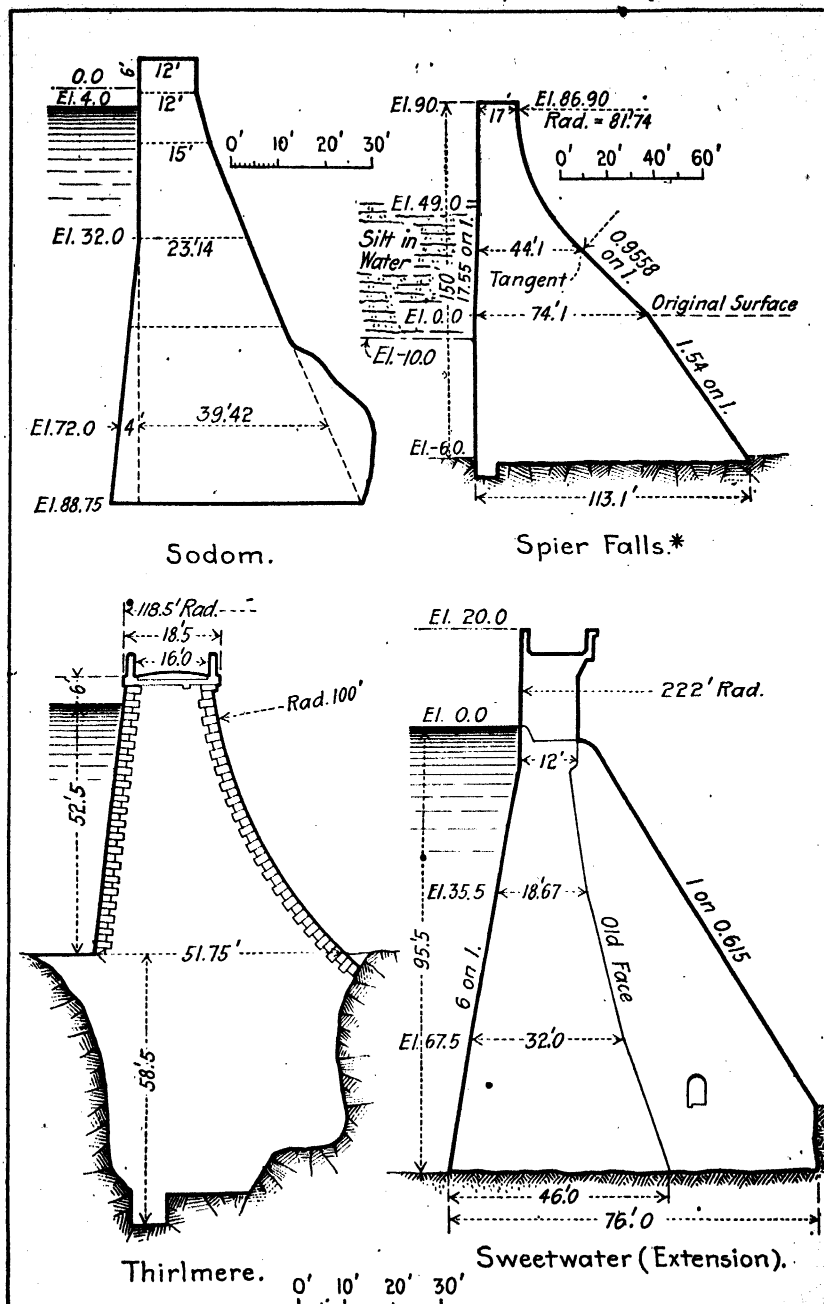


FIG. 74.—Non-overflow dams.

* For overflow portion, see p. 174.

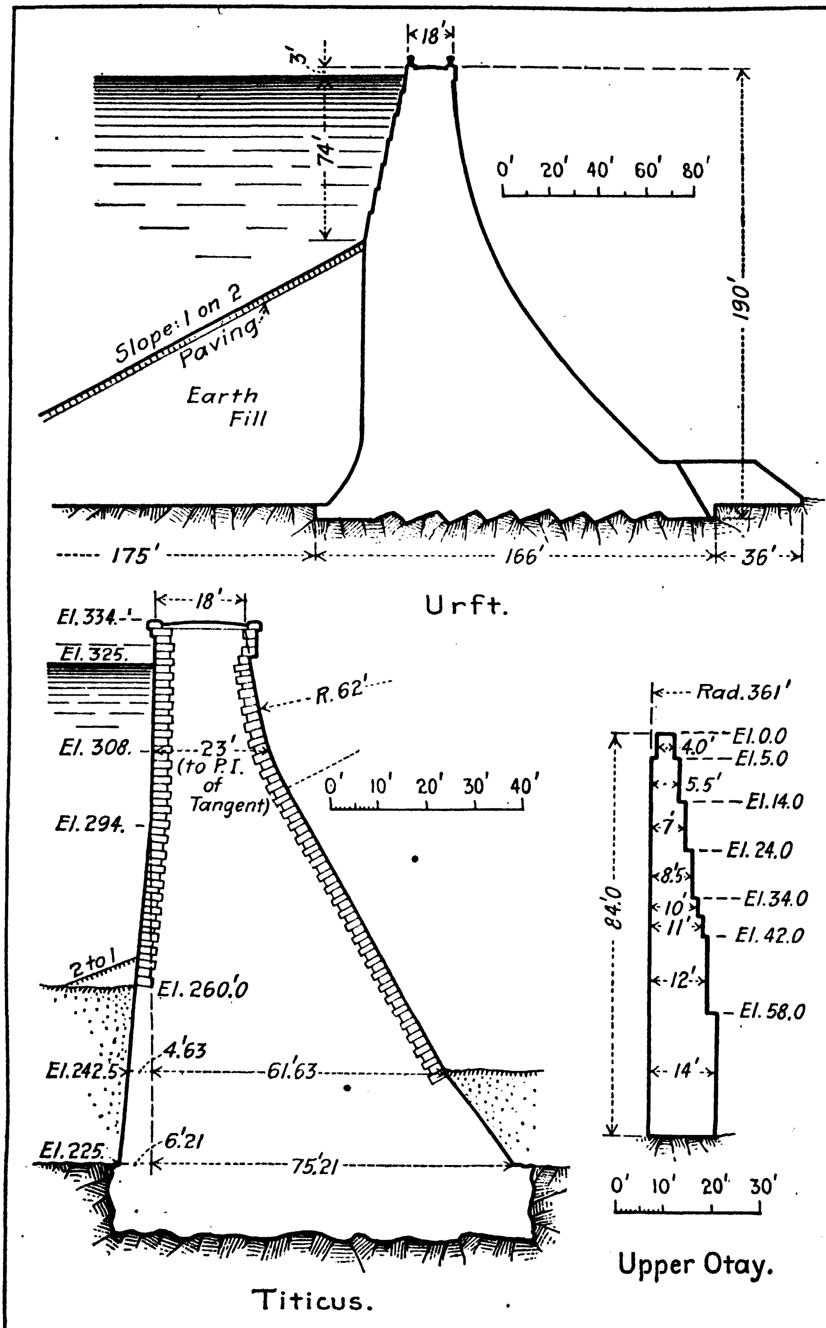


FIG. 75.—Non-overflow dams.

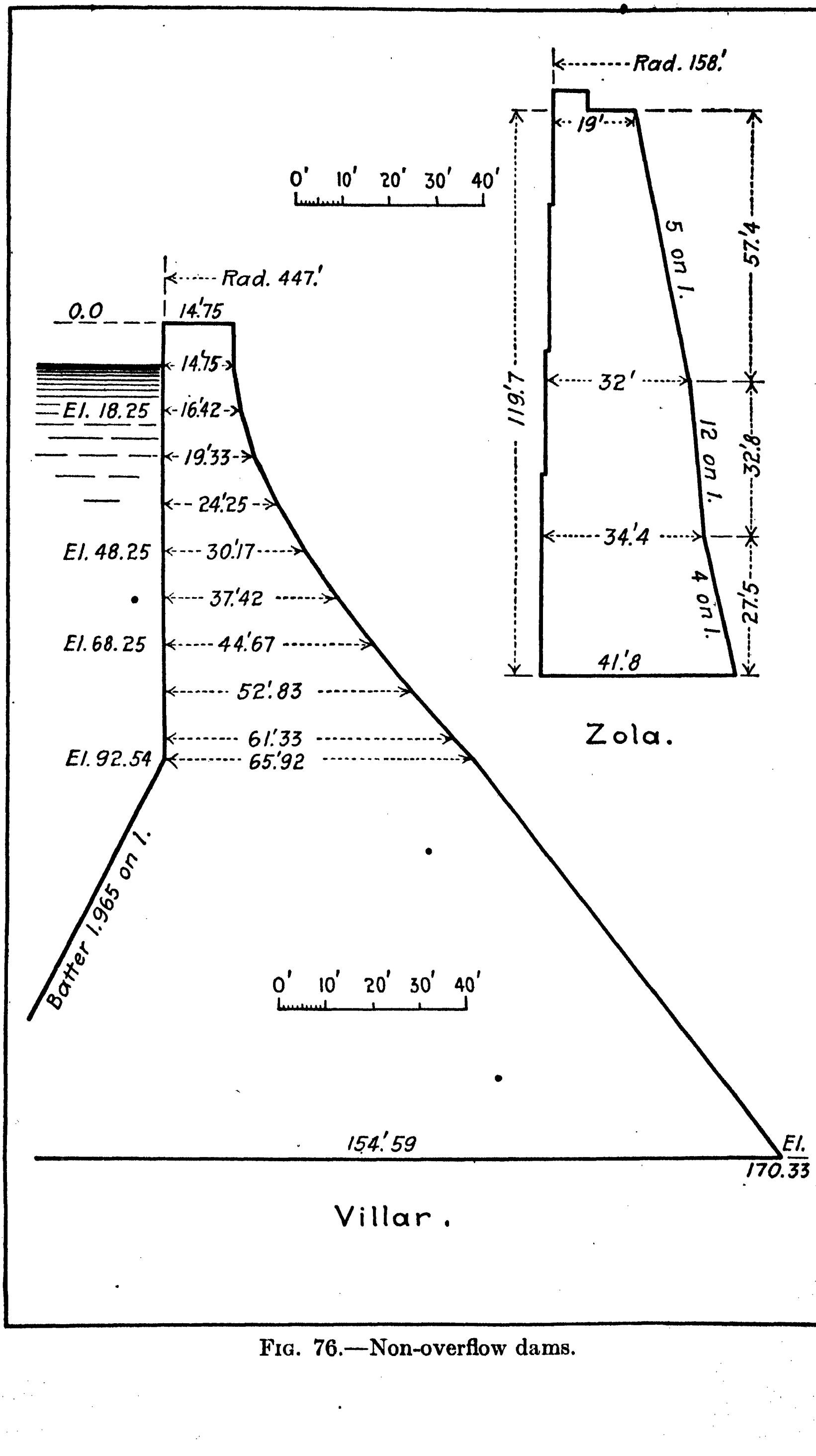


FIG. 76.—Non-overflow dams.

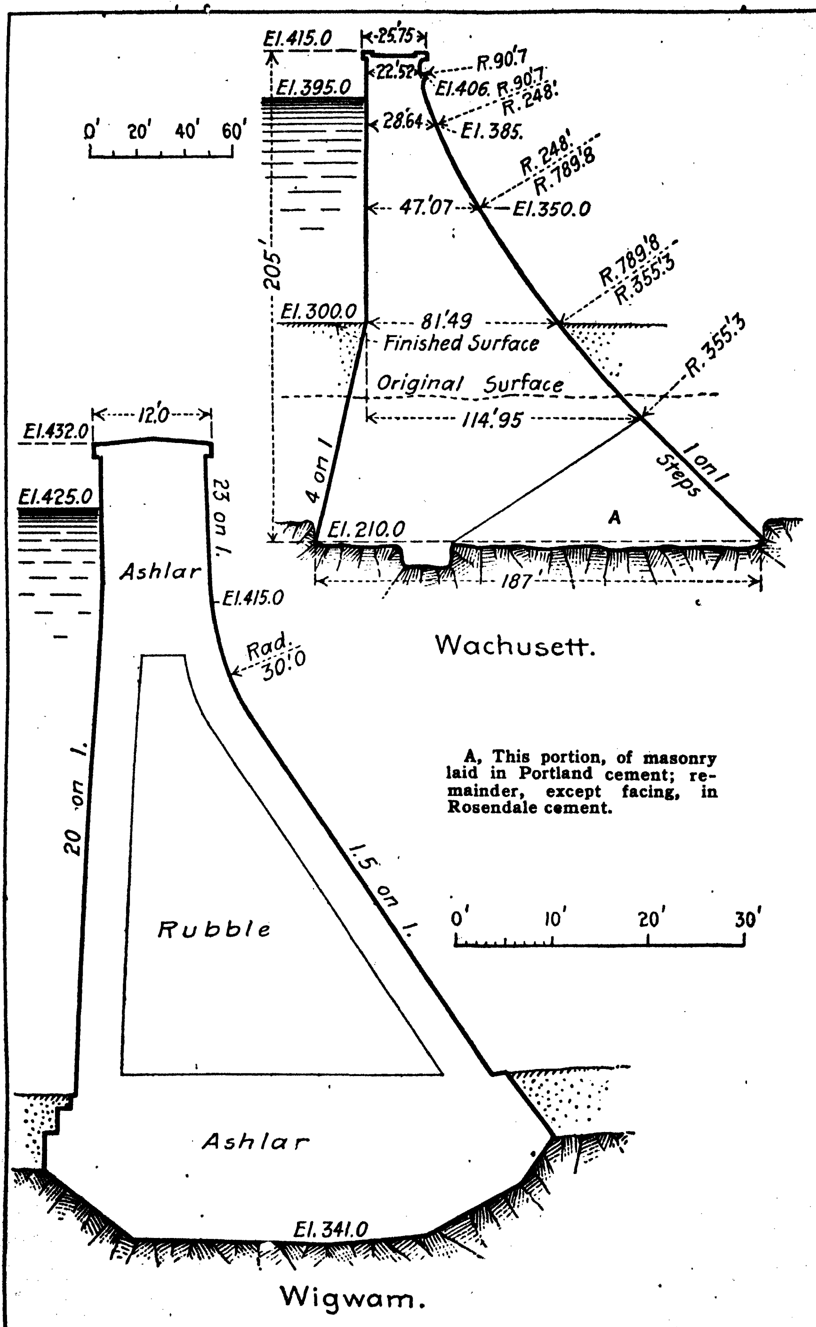


FIG. 77.—Non-overflow dams.

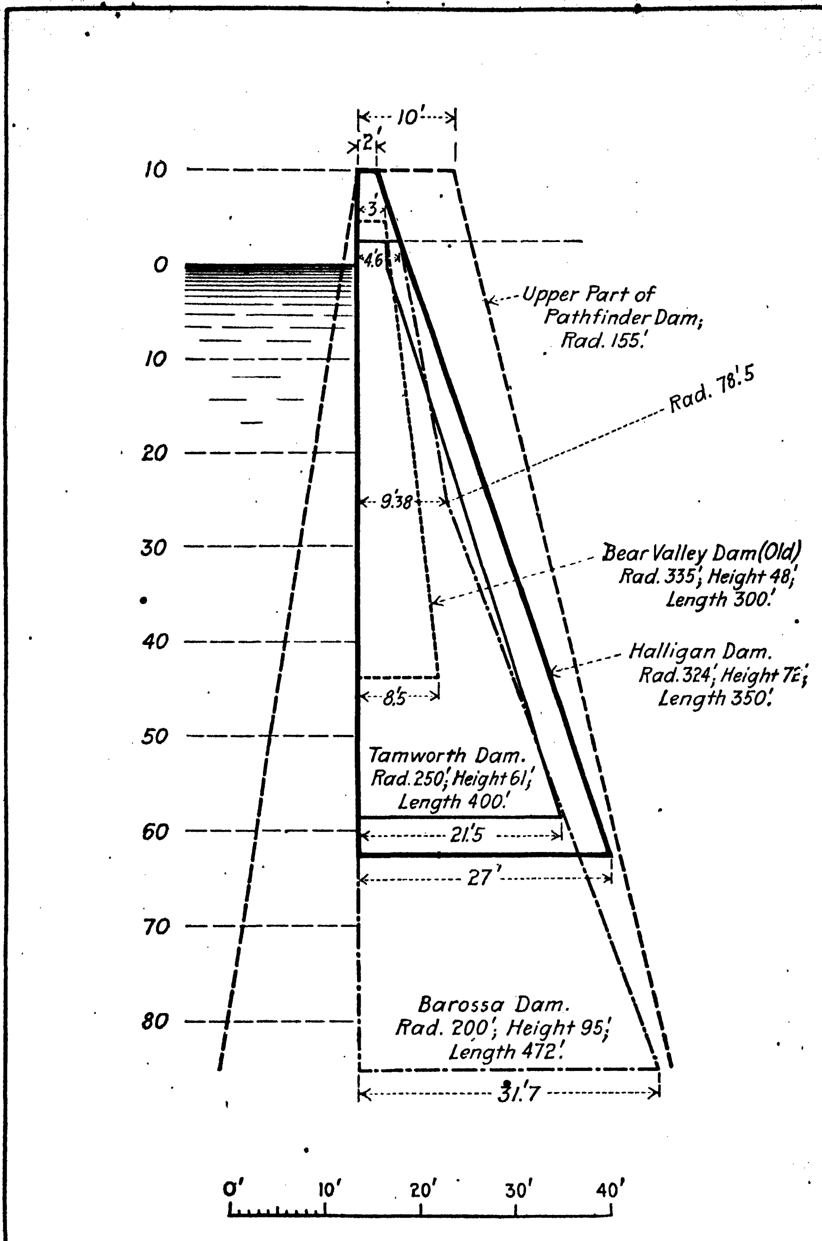


FIG. 78.—Comparison of sections of arched dams.

Radii of horizontal curves are to upstream faces. (T. A. S. C. E., Vol. 75, 1912, p. 127.)

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(See also p. 146 and p. 157. For partial list, see Davis and Henny, *Paper* No. 47, Int. Eng., Congress 1915, p. 710.)

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CHAPTER 8

ROCK-FILL DAMS

Advantages. A rock-fill dam is an embankment of loose rock fragments supporting a watertight face or containing a watertight core of masonry, wood, or metal, or some combination of these materials, or backed by an earth embankment. Chief advantages are low cost and quick construction. Where the foundation is pervious rock, making an earth dam impossible and requiring a masonry dam of large mass to resist upward water pressure, the free-draining character of a rock fill makes it especially suitable. Almost complete elimination of cement is a great advantage in inaccessible regions. By changing the shape and proportions of the dam, it is adaptable to a wide range in topographical conditions. In Italy it is coming into favor because of resistance to earthquakes.[1] In India, rock-fill dams have been used for a long time. There are many in the United States.

Disadvantages. (1) Large proportion of voids is a danger if dam is overtopped, as they afford passage to a flow that might dislodge the rock fill. (2) The inevitable settlement may rupture the watertight element. (3) Failure follows overtopping, as at Gohna Lake dam.[11] (4) Leakage is greater than through masonry dams. (5) Can be used only where rock is available. (6) Prospecting to locate leaks may endanger the structure.

Requisites. (1) Availability of quarrying rock. (2) Since the gross foundation pressures approximate those of masonry dams, there should be unyielding foundations which cannot be eroded by copious seepage. (3) Abundant weight downstream from core to resist the water pressure, in the ratio of at least 3.5 to 1, based on 40 per cent. voids.[3] (4) Place as much as possible of the rock fill downstream from the watertight membrane. (5) A watertight core or facing of permanent materials, so constructed, supported, and protected that it cannot be disrupted. (6) Stones for the fill of good weathering qualities, freed of earth, strong enough to take the superposed weights, and so placed that very little settlement can occur. (7) Ample spillway capacity; further protection is advocated by some engineers by paving the downstream slope to serve as overflow under small surcharge without failure; the economy of this in contrast to enlarging the spillway is questioned by Jorgensen.[3] Inadequate spillway is held accountable by some for the Lower Otay dam failure.

Details. Watertight member, whether core-wall, diaphragm, or facing, should be carried down into trench in solid rock. Metal core or facing is usually of steel plates riveted and calked, coated with asphalt, or asphalt and burlap, and supported by concrete walls or embedded in concrete. Plates should not be less than ¼ in. thick and ought to be thicker in the lower part of a high dam. (Used on Lower Otay, see p. 191.) Figure 79 illustrates a

reinforced concreté face. Settlement of reinforced-concrete water face on Strawberry dam[4] was provided for by a sliding watertight apron, moving between ribs cast into the hand-laid dry fill at 60-ft. intervals. Timber is sometimes used for the watertight member either permanently or until settlement is attained; the Cucharas[5] dam was made tight by placing a 5-in. creosoted timber facing over the leaky concrete. Where tightness is not essential and boulders are not obtainable, as at Omega Mine, Cal.,[6] a dam may be constructed of units composed of wire mesh baskets filled with large gravel and rocks, units 1 by 2 by 8 ft., placed lengthwise with the stream; such dams would not be permissible in many situations.

Top width should be not less than 10 ft., and height above extreme high water not less than 5 ft. Upstream slope may be 1 on ½, or even steeper if laid up as a dry rubble wall to support watertight facing. Investigate tendency to slide with reservoir empty. Downstream slope, or both slopes if core is used, may be 1 on 1 to 1 on 1½, and may be rough or laid to a neat face. Some rock-fill dams have downstream slope steeper than the angle of repose of the material; this is questionable practice. Others have nearly vertical faces, and some have stepped faces of roughly squared stones. One or both faces may be laid in mortar, or faced with concrete.

Practice. M. M. O'Shaughnessy[2] disapproves sheet steel in the concrete facing of a rock-fill dam, as adhesion of the concrete to the smooth face of the steel would be questionable, and, with reservoir empty and no expansion joints, temperature changes might cause the steel to buckle, thus creating a cavity which might be subjected to hydrostatic pressure. Soil, silt, and clay should be excluded from the mass of the dam, but not quarry waste, composed of spalls. If there is a shortage of spalls, break off sharp edges of flat stones, and chink in the cavities with broken stone. A rubble wall for the downstream slope, or the hand placing of rock in this portion of the dam, is an unnecessary refinement and expense, except for esthetic purposes. The object of reinforcing the concrete facing is to prevent cracks, so that each section will be a unit slab, firmly attached to the stone masonry, but free to move at the joints in response to temperature or settlement. Anchor rods through the masonry form an effective bond between it and the reinforcing rods, and are also useful in construction in fixing accurately the position of bars.

Flood Flows. Escondido dam in 1916 was overtopped for 6 hr., to a depth of 12 in. at center and 2 in. at abutments. Settlements up to 1 ft. occurred, but the dam did not fail.[7] Morena dam[2a] was not overtopped in the 1916

Table 51. Rock-fill vs. Solid Masonry. Comparison of Roosevelt and Morena Dams.[2b] (Both Built before 1912)

	Roosevelt dam (solid masonry)	Morena dam (rock-fill)
Height	284 ft.	267 ft.
Thickness of base	170 ft.	300 ft.
Thickness of top	16 ft.	16 ft.
Contents	340,000 cu. yd.	306,000 cu. yd.
Crest length	1080 ft.	550 ft.
Time consumed in building	4 yrs.	5 yrs.
Cost	$3,468,000	$1,100,000

flood; the reservoir filled to within 18 in. of crest without damage—a testimony to the stability of a rock-fill dam, properly tied into the abutments. The radial spillway gates became choked with debris and failed to function completely, as a result of which the prevailing public hysteria compelled their removal; the designer* was not consulted. *Failure of Lower Otay*[9] *dam* occurred Jan. 27, 1916, after 15 days' run-off from the watershed had raised

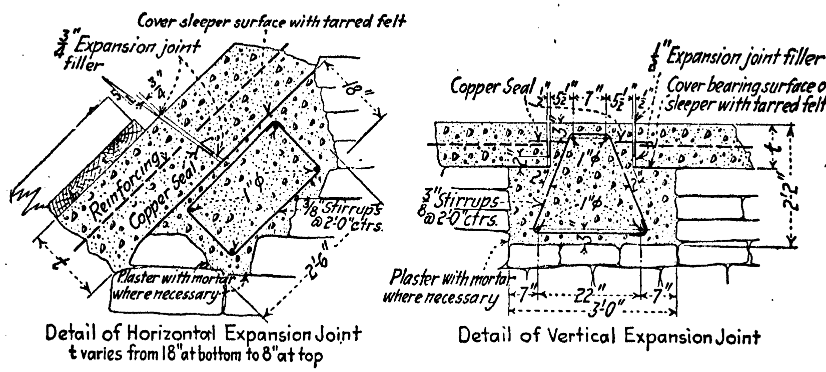

FIG. 79.—Expansion joints, facing of Dix River dam.

reservoir level 36 ft. At time of failure, depth of overflow of 4 to 6 in. over top of dam was withstood for 7 hr. Total depth of water behind dam was 130.8 ft., in contrast to previous maxima of 109.5 ft. in June, 1909, and 119.5 ft. in May 1910. Although completed in 1897 to withstand a head of 124 ft., never had there been a saturation of the rock fill until 1910, when there resulted a slight settlement on the upstream side.

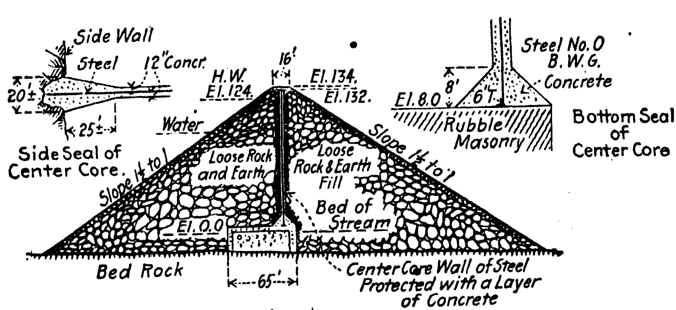

FIG. 80.—Lower Otay dam.[2c]

Dix River Dam, Kentucky Hydro-electric Co., is 270 ft. high, with the watertight face upstream. Adjacent rock fill is hand placed to assure uniform support. Face is reinforced concrete slabs, 18 in. maximum thickness and 8 in. minimum, 48 ft. wide between vertical expansion joints. Figure 79 gives details of expansion joints. Horizontal expansion joints are at the 50-ft. levels. Leakage cracks resulting from inevitable settlement of the face are reduced by an excess of reinforcing steel to restrict the opening. The reinforced-concrete

* Correspondence with M. M. O'Shaughnessy.

face is protected by a 3-in. layer of tongue-and-groove planks, with long sides horizontal, held to position by wales on 8 ft. centers, secured to the masonry by ⅞-in. bolts on 4-ft. centers. This timber facing affords the first line of defense against leakage since timber will deflect to a greater degree than concrete without leakage. It is provided only below minimum water level so that it will be saturated always. (Notes from L. F. Harza, Engineer.)

Settlement Measured. Strawberry Dam.* This dam is 139 ft. high above stream bed, and has a top length of 587 ft. It was constructed in 1915. Measurements made in September 1917 to determine the settlement and horizontal movement, are given in Table 52.

Table 52. Record of Settlement, Strawberry Dam, Feet

Station	Vertical settlement	Horizontal settlement
0	(End of Top)	
1 + 50	0.94	0.74
+ 70	1.00	0.70
+ 90	1.06	0.66
2 + 10	1.03	0.80
+ 30	1.12	0.80
+ 50	1.28	0.86
+ 70	1.27	0.88
+ 90	1.25	....
3 + 10	1.25	....
+ 30	1.24	0.74
+ 50	1.22	0.66
+ 70	1.14	0.68
+ 90	1.10	0.56
4 + 30	0.67	0.66
+ 50	0.55	0.41
5 + 87	(End of Top)	

These denote the total changes after 2 years, and after the reservoir has been twice filled substantially full. Water stood 47 ft. below the crest and 80 ft. above the stream bed when these measurements were made. (Information from M. M. O'Shaughnessy.)

Bibliography, Chapter 8. Rock-fill Dams

1. Int. Eng. Congress, 1915, *Paper* 40, p. 544. **2.** *T. A. S. C. E.*, Vol. 75, 1912. **a**, p. 27; **b**, p. 30; **c**, p. 66. **3.** *E. N.*, Mar. 9, 1916, p. 462–4. **4.** *E. R.*, Aug. 6, 1916, p. 261. **5.** *E. N. R.*, Aug. 1, 1918, p. 229. **6.** *E. N. R.*, Apr. 12, 1917, p. 103. **7.** *E. N.*, May 25, 1916, p. 1006. **9.** *E. R.*, February, 1916, pp. 191 and 225. **10.** *E. N. R.*, Nov. 22, 1917, p. 988.

* Described in *E. N.* Mar. 30, 1916, p. 605.

CHAPTER 9

EARTH DAMS OR DIKES

Where Used. Earth dams or dikes are used where good rock foundations either do not exist or are at such depths as to make excavation for masonry impracticable, and where materials of desirable qualities are available within a few miles of the site. Sometimes a combination of rock fill and earth is used, as at Snake River.[66]

Merits. Great permanence and security; it is common knowledge that, in rigorous climates, earthworks are the most lasting structures built by man The apprehension of the layman is allayed. This type was chosen at Calaveras on basis of economy and resistance to earthquake damage, as exemplified at San Andreas.[1] Economy depends upon the availability of proper materials.

Requirements.* Water cannot be allowed to pass over the top. Provide ample freeboard against extreme flood and wave action, and a spillway of liberal capacity. Pave the area subject to wave action. Since earthy materials always permit seepage, they must be so compacted as to reduce it to a non-eroding rate, and there must be a water stop—either core wall or slope paving. "To be safe for an indefinite period, an earth dam should have somewhere in its cross-section a durable element, or member of construction, capable of resisting attacks of burrowing animals, not infrequently experienced from upstream as well as downstream side."[2]

Classification.† Four classes of earth dams may be distinguished for convenience: (1) simple embankments, substantially homogeneous; (2) embankments with core; (3) embankments with facing; (4) hydraulic fills.

Homogeneous Embankments. Class 1 should be limited to dams of moderate height, in locations where the consequences of failure are not serious, and where loss from seepage is of minor moment. Many homogeneous dams are in successful use in India, where this type is preferred to puddle core.[3c] Tabeaud dam, Cal., is of this type. Watertightness as well as stability against slips can be approximated only by proper selection and consolidation of materials. Best material is a heavy, naturally impervious, tenacious earth, such as hardpan, unmodified glacial drift, or bowlder till. Indian practice[3a] now uses 1 part clay and 1 part shale. Sand, or fine gravel and sand, have been used; initial leakage was high, but the embankment eventually silted tight. Consolidation is important; an unrolled embankment near Trinidad, Colo., allowed water to pass through, and partial failure resulted.[4]

DAMS AND EMBANKMENTS WITH CORES

Purposes of cores are: (*a*) lessening of saturation of downstream slope, thereby promoting cohesion and stability of the material; (*b*) minimizing

* See also "Design," on p. 202.
† See Holmes, *E. N. R.*, June 26, 1924, p. 1102.

erosion if dam be slightly overtopped; (*c*) prevention of flow through the pervious strata beneath the dam; (*d*) resistance to burrowing animals; (*e*) interception of accidental pervious streaks, which might by percolation be developed into water channels and eroded to destructive dimensions. A dam near Waterbury, Conn., constructed in 1868 without a core leaked several hundred thousand gallons per day, with full reservoir. This loss and anxiety for stability led to reconstruction with a core wall in 1921.[5] Whatever kind of core be used, precautions must be taken to prevent water under any possible head flowing down along the side of the core and beneath it. Dams of Miami Conservancy District utilize reservoir blankets rather than core walls to curtail seepage.[14h]

Position and Materials. Cores are usually vertical walls or diaphragms at or near the center of the embankment, extending from a trench in impervious material beneath the dam to full reservoir level, or above (see also p. 198); sheet piling, loam, puddle, or masonry have been used.

Sheet Piling. If the purpose is to retard flow through pervious substrata, the length of the piling should be a function of the head. See studies on p. 204, and tests by J. B. Hays[6] and J. B. T. Colman.[7] Sheet piling is used where the dam site is overlaid with pervious sand and gravel, free of boulders, or nearly so, and of such depth or so wet that a core-wall trench cannot be excavated economically to an impervious stratum. Extreme care must be exercised to keep adjacent piling in contact throughout; large gravel or boulders prevent close driving and defeat object. Top of piling, in the trench, should be thoroughly embedded in the bottom of the masonry, puddle, or loam core. Piling may be extended to full height as the only core; but for several reasons including uncertainty as to permanence, sheet-pile cores should not be used in important permanent dams above permanent lowest ground-water level. Chief disadvantage is inability to observe final position of sheet piling.

Wooden Sheeting. Miller sheet piles were used in portions of North dike of Wachusett reservoir; where length exceeded 30 ft., piles were built up of three thicknesses of 2-in. spruce planks, spiked together. The outer planks were wider than the inner, and grooves were formed on both edges, into which splines fitted. Planks jetted to position; lengths, 45 to 67 ft.; best 10-hr. day's work was 24, averaging 16½ in. wide and 46 ft. long. Piles less than 30 ft. long were single 4 in. planks. On Keechelus dam[8] in a particularly wet stretch, 9-in. by 12-in. Wakefield piling of Douglas fir was substituted for the concrete core wall. Table 58 indicates other dams having timber cores.

Steel piles have the advantage over wooden sheeting that the interlock will generally keep the piles in contact during driving. The core of the Sevier Bridge dam, Utah,[9] consists of 38-lb. U. S. steel sheet piling, 20 to 40 ft. long, with tops at elevation of river bed. To the tops were attached 4 by 10-ft. plates of copper-bearing Keystone steel, No. 10 gage, fabricated by American Bridge Co., lapped 1½ in., and riveted with ⅜ in. rivets. At intervals of 50 ft., U-shaped expansion joints were inserted. Besides the shop coat of red lead, the sheets were given two coats of asphaltic paint, applied hot. The

impounded water is strongly impregnated with lime, which forms a protective coating on metals, and led Chief Engineer C. S. Jarvis to substitute steel for the ingot iron called for in plans. Metal cores have been used in rock-fill dams, notably the Avalon and Lower Otay (see p. 191). Steel piles have also been used as support for the trench of the concrete core wall, and eventually left in place. Such construction was used on the Milton[10] and Wanaque dams.[11]

Loam cores should be started in relatively wide trenches excavated to rock or into impervious earth. If rock is seamy, the seams should be cleaned out and thoroughly filled with concrete or grout; pervious rock should be grouted or plastered with rich cement mortar. To prevent water passing along the rock surface beneath core, low concrete, rubble, or brick cut-off walls should be built wherever necessary, to extend up into the loam. Loam* should be very fine grained, free from stones, roots, and vegetable débris; it should be spread in thin horizontal layers (3 to 6 in.), and very thoroughly compacted. It should be forced tightly against the sides of the trench and carried above, with liberal thickness, in the embankment. If the rock on which the core is to be founded is seamy and has only shallow cover of earth, it is sometimes necessary to uncover a wide strip on the reservoir side of core. After scrupulous cleaning, this rock should have all seams grouted or otherwise thoroughly stopped, and its whole surface, if pervious, covered with rich cement mortar or grout.

At Providence,[12] soil core was preferred to concrete for following reasons: (1) Materials available were of a dependable uniformity. (2) Instability of a core wall of the dimensions economically practicable, under unbalanced pressures, raises question of its watertightness. (3) Presence of wall interferes with placing of watertight material. It was concluded that design adopted involving two and one half times the impervious material of the core-wall type would produce a tighter structure and at less cost. A concrete blanket intercepts flow beneath loam core.

Puddle† cores should be of carefully selected materials, faithfully mixed and compacted; should start in trenches excavated to rock‡ or into impervious earth. Rock surface should be cleaned and seams made tight.

British engineers favor clay puddle cores rather than masonry. Strange claims greater watertightness and flexibility for the former.[3c] The core is placed along the center line of the dam, vertically over the puddle trench.

Dimensions. Cores should be carried 2 to 4 ft. above full reservoir level, and have a top width of about 5 ft. with side slopes of 10 on 1 to 1 on 1. With slopes approximating 1 on 1 the puddle core really is the dam; the outer portions of the embankment serve principally to protect the puddle from waves, frost, etc. Thickness necessary for puddle core depends largely upon character of the other materials in the embankment, and conditions and methods of construction; at the base of the dam it should rarely be less than one-third the depth below full-reservoir level; below the ground surface, in the trench, the thickness may be materially reduced. Rankin states that

* These provisions apply as well to impervious embankment, see p. 207.

† For data on puddle *per se*, see p. 803. For dimensions of notable earth dams with puddle core, see *E. C.*, Jan. 18, 1911, p. 90.

‡ At South Haiwee dam, Los Angeles, clay cut-off, 8 ft. thick, extends to a maximum depth of 120 ft.[14a]

thickness of base of puddle wall should be about one-third the height, and thickness at the top about one-half the base.

Some engineers prefer to step the sides of the puddle core, or to step the reservoir side and make the other substantially vertical.

Position. A puddle wall is used in three positions: (*a*) in the center of the dam; (*b*) on the upstream slope; (*c*) between these two. The advantages of (*a*) are: (1) vertical continuation of the puddle trench; (2) minimum quantity; (3) protection from disintegration by wind, water, burrowing animals, etc. The disadvantages of (*a*) are: (1) distortion or rupture by settlement; (2) difficulty of repairs; (3) upstream embankment may be saturated to an undesirable extent. The advantages of (*b*) are: (1) settlement is uniform with upstream slope; (2) ease of repair; (3) constitutes a waterproof covering. The disadvantages of (*b*) are: (1) more material required than for (*a*); (2) sub-soil water may saturate the dam; (3) exposure to action of waves, sun, and burrowing animals; (4) lack of stability at ordinary slopes of dams. A layer of puddle has sometimes been extended from the core, beneath the embankment on the reservoir side and over a portion of the reservoir bottom, to aid in preventing percolation beneath the dam.

Materials. Clay was formerly preferably used as a core. The pressure of surrounding masses forcing out surplus water, leaves clay in such condition that it will neither absorb water nor allow percolation. Surface clays make better cores than deepseated clays. Clemens Herschel claims that pure "binding" gravel makes a more waterproof stop than clay.[2] Davis and Henny recommend a well-blended mixture of gravel, sand, and clay.[3b] Puddle, free from lumps or stones larger than 2 in., if laid in 6-in. layers, will be practically watertight. The test for puddle dams used in India is to dig a hole 2 ft. square through 1 week's work, fill with water, and note the time it takes to disappear. Good puddle, 4 days after depositing, should cut like cheese and show no air spaces.

Protection. Both puddle and loam cores must be protected from the inroads of burrowing animals. A layer of broken stone, 6 in. thick on the lower slope, has been found a sufficient protection against vermin in India. Puddle of 80 per cent. gravel and 20 per cent. clay is less easily attacked than pure clay.[2] Throttle dam[13] near Raton, N. M., has a 12-in. paving, but the puddle wall is additionally protected by a galvanized, corrugated, ingot-iron sheet of No. 12 gage. See also p. 109.

Concrete Cores. Rubble walls were formerly used extensively, but recent practice favors a rich Portland cement concrete core; this best fulfills all functions. Care should be taken in placing to avoid honeycombed and porous spots. The watertightness of old core walls has, in numerous cases, been increased by plastering the upstream faces with rich cement mortar. The use of a concrete core reduces the problems of design of masonry culverts or outlets, as these can be made integral with it. Masonry cores have been extensively used, notably at Sudbury, Croton Valley dams, and Ashokan dikes.

Depth of trench depends on the function of the core wall. The effect of this construction on the hydraulic gradient of percolation is outlined by Justin.[14b] Wall should start on clean sound rock, or in a trench in very compact impervious earth, or on sheet piling extending into impervious material. If a good

foundation for a core wall exists 10 ft. or somewhat less below the stripped surface, there appears no good reason for carrying it deeper. At Keechelus dam, core wall was successfully founded on earth.[8] It is character of foundation rather than depth which is important. In some cases, however, because of porous materials, it has been necessary to extend cores to great depths, some in excess of 100 ft., notably at Wanaque dam where a core 20 ft. wide was carried down a maximum of 90 ft., trench being maintained by sheet piling.[11] On Tieton dam, 100-ft. depth was attained by shafts, cross tunnels, and stoping upward.[15]

Trench and Foundations. If practicable, in the foundation trench, the concrete should be compacted against the well-trimmed sides. In loose, running earth it will be necessary to give the trench wide flat slopes, and start the wall forms on the rock; the trench should be of liberal bottom width and most carefully refilled with compacted embankment material. If trench has been sheeted and braced, as much bracing as feasible should be removed as concrete is placed. No wooden or hollow metal braces should be left to pierce the concrete transversely. Concrete should be packed tightly against any sheeting which must be left in place. If the concrete core wall goes to rock of reasonably good quality, it is better not to disturb the rock by blasting. At Weston reservoir, F. P. Stearns excavated into rock a very little where rock was loose, or where there were large seams, but the rock was cleared for a distance of 20 ft. upstream from the core wall, all seams filled with mortar, and subsequently covered with an embankment of fine, clayey earth; this 20 ft. was about equivalent to the expected water depth in reservoir; Stearns advocated this as a good rule. At Phelps Brook dam,[16] Hartford Water Supply, the ledge for a distance of 10 ft. upstream and for 3 ft. downstream from the core, was covered with a blanket of concrete at least 3 ft. thick. If rock has approximately horizontal seams through which water can flow, a trench should be excavated and the wall extended downward as a cut-off. Grouting under pressure should be done in holes drilled in the bottom of the cut-off trench if seams which would cause trouble are suspected at depths too great for excavating trench at reasonable cost. Importance of dam must determine extent of such precautions. At Henshaw dam[17] 2-in. holes, on two lines 6 ft. center to center, were put down 20 ft. and grouted to 100 lb. pressure. At Milton dam,[18] delay in construction was avoided by embedding 3-in. pipes in the core at 5-ft. intervals, through which grout holes were drilled at leisure by a single Calyx shot drill.*

Position. Core walls are sometimes placed upstream from the center line to reduce the amount of saturated embankment. If a core wall in this position is carried above water level, it will intersect the upstream slope, and serve as protection against wave action. On Milton dam, Ledoux placed core wall at upstream toe, and made it continuous with the upstream paving.[18]

Dimensions. A thin, core wall in a high earth dam is not advisable; unbalanced pressures due to different classes of fill on either side, to varying degrees of pressure as state of saturation changes, and to varying settlement of the embankment produce unequal loadings. Cores are in many cases built too

* See grouting methods at Estacada dam, by Rands, in *T. A. S. C. E.*, Vol. 78, 1915, p. 447; and "The Use of Grout in Cut-off Trenches and Concrete Core Walls for Earthen Embankments," by H. J. F. Gourley, *Trans. Inst. Water Eng.*, Vol. 27, December, 1922, p. 142.

light. Concrete core walls may be carried to heights of many feet ahead of the embankments, but if this is done, the embankments must be built simultaneously on both sides with difference of level not greater than a few feet; otherwise the walls might be pushed over. The stress at top of a core wall is insignificant compared to stresses at greater depths. Some recommend keeping top width to 2 ft., and using material saved for increasing the bottom dimensions;[2] top width of a concrete core wall should be 1½ to 5 ft., according to importance of the dam. Top should be at full-reservoir level or 1 to 5 ft. above. Each face should batter 20 on 1 to 10 on 1, or may be stepped one or both sides. Thickness at the level below which trench is filled solid with masonry should be one-eighth to one-sixth head. E. Sherman Gould's rule was to give a core wall a bottom thickness equal to one-quarter the greatest depth of water expected at the dam, and to draw in with offsets of 10 in. on each face every 10 ft. in height.

Construction. Longitudinal grooves (or keys*) are essential in top surface of concrete placed any day. On Ashokan dikes a single center groove 3 to 6 in. deep, and 6 to 12 in wide, was used. Three keys were used on Phelps Brook dam;[19] in the vertical joints, ingot-iron waterstops were used. These joint surfaces must be scrupulously clean, free from laitance, and thoroughly wet when fresh concrete is placed against them. If bolts through the wall are used to support the concrete forms, such bolts should, preferably, be removed, or at least portions for 2 in. in from face, and the holes thoroughly filled with cement mortar. A well-built concrete core should not require plastering.

Priest dam,[20] *Hetch-Hetchy project,* has a thin core wall of concrete, with horizontal joints 16 ft., and vertical joints 50 ft , apart, to permit conformation to settlements in the dam without loss of watertightness. Joints are tarred to prevent bonding, and are made watertight and flexible by strips of 10-gage copper. The wall is 2 ft. thick at top: batters of 1 to 84 result in a thin section. To strengthen it against unbalanced construction stresses, each section between joints is guyed temporarily by cables leading to deadmen set in the embankment.

Reinforced-concrete core walls have been used on many dams by U. S. Reclamation Service, the thickness varying in general from 12 in. at top to 4 or 5 ft. at ground line. Notable dams with this type of core are Pathfinder, Wyo., and Strawberry River, Utah[21]

PAVING†

Substitute for Core Wall.‡ Embankments with facing are much less common than those with cores. A few dams have been so built, notably at Milton, Ohio, for Youngstown Water Supply.[18] In general, such construction should be limited to artificial reservoirs largely in excavation. Facing may be concrete, rubble in mortar, brick, or puddle; it should be laid on freshly trimmed slopes and ample precautions taken to prevent settlement, cracking, and damage by ice, waves, or extreme summer heat. Puddle must be such

* *A raised key,* or tongue, is preferred to a groove by some engineers as being easier to make clean On a horizontal surface especially, cleaning is easier These remarks and those below apply to much concrete work other than core walls, in which interruptions are necessary.

† For reservoir use, see p. 538.

‡ For function, see p 193.

that sliding and buckling cannot occur. If water in reservoir be rapidly drawn down, there is danger that back pressure of water in the embankment may lift the facing. Provision for this must be made by drains or weepers, which will not in turn become danger points in other ways.

Protection of slopes by paving or riprap on the upstream side, and covering with grass, other suitable vegetation, gravel, or riprap downstream, is required for the integrity of the structure (see p. 112). In arid countries grass is impossible. Some engineers keep all vegetation off dam to prevent burrowing by rodents for the roots. In some places, close matting roots will grow better than grass. On the water slope, paving or riprap (termed "pitching" in British practice), should extend from bottom (unless reservoir is never to be drawn down), to at least 2 ft. and more vertically above full reservoir level; it should always extend high enough to protect the slope against erosion. Many dams have the paving 6 to 10 ft. higher, depending on height of waves.

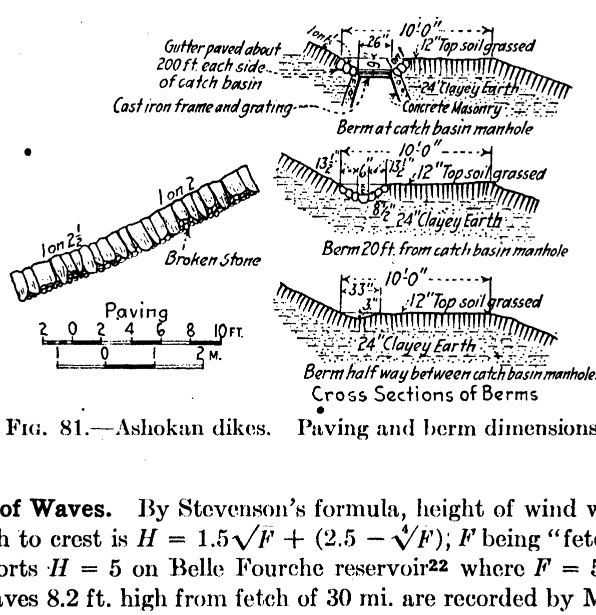

FIG. 81.—Ashokan dikes. Paving and berm dimensions.

Height of Waves. By Stevenson's formula, height of wind waves in feet from trough to crest is $H = 1.5\sqrt{F} + (2.5 - \sqrt[4]{F})$; F being "fetch" in miles. Henny reports $H = 5$ on Belle Fourche reservoir[22] where $F = 5$. On Lake Geneva, waves 8.2 ft. high from fetch of 30 mi. are recorded by Mead.[23] On Ashokan reservoir, with wind velocity of "probably 120 mi. per hr.," Thaddeus Merriman records waves 12 to 15 ft. high.[14c] A wind reported to have attained a velocity of nearly 100 mi. per hr. along a reservoir 5 mi. in length, drove waves over the top of a masonry dam rising 20 ft. above the water surface, also above the top of heavy stone slope paving on adjacent earth dams (slope 1 on 2) with such force as to cut the slopes and do damage to the paving. This incident also showed that stones of large area, laid close together with joints tightly spalled, supported on a deep bed of cobbles and stone fragments, having large voids, were disturbed by the water held in the voids, which could not escape rapidly enough as the waves receded. Paving on such a bed should have very free vent at its surface. An 80-mi. wind drove sheets of water over the top of Haiwee dam when original water surface was 20 ft. below the top. Reservoir, 6.5 mi. long.[63]

Water Slopes. On water slopes, pavement is usually of stones, but concrete in slabs or blocks and rubble masonry in mortar are used in some instances, especially in distributing and equalizing reservoirs or sedimentation basins which are to be cleaned occasionally.* Oil linings,† formed by applying 1 to 3 gal. of heavy oil per sq. yd. to the earthwork, have been used on some canals and reservoirs in California.[25]

Stone paving can be laid dry and grouted, as at Milton dam,[18] where results equal to masonry laid in mortar were obtained. Stone paving varies from large rectangular granite blocks, with 1-in. joints well filled with tightly driven spalls laid on gravel or broken stone, to stones of miscellaneous sizes and shapes only roughly laid to slope. Materials at hand, permissible expenditure, nature and importance of dam, and similar considerations determine. Likewise thickness of paving may vary from 1 ft. to several. Stones should be hard and of good weather-resisting qualities, especially in regions having severe frost. Heaviest and best stones and greatest care in laying should be used from top of paving to a few feet below full-reservoir level, in order better to resist waves and ice. For this belt, paving of a distinctly better class is frequently used, and cheaper paving or riprap below. For best results, paving should be laid on a layer of broken stone or gravel from 6 in. to 1 ft. or so thick, to prevent earth being washed out from between the large stones by waves or by water draining out when the reservoir level is rapidly lowered. To resist sliding, or displacement by ice, it is well to distribute many long stones in the paving, like headers, extending through the backing. Longest joints should be parallel to axis of dam. Berms in long, steep slopes reduce tendency to slide. Always place a berm at lower limit of paving. If riprap or a poor class of paving be used, the thickness should be greater in the belt at and near full-reservoir level, since by action of ice and waves some stones will be displaced or moved toward the bottom. Caution must be exercised, however, not to place too heavy a load on the upper portion of a slope if the bank be composed of material which, when saturated, may slump under the load. On the other hand, slumping of a bank or an excavation slope in fine, water-bearing material may be stopped or prevented by loading with a thick layer of coarse stone; in this case the layers should be thicker toward the bottom. Thick riprap is cheaper and better than paving if large stones can be had plentifully at small cost; it stops waves from gliding up the slope and does not require expensive repairs when displaced. On flat slopes (1 on 4 or flatter), riprap of small stones is thoroughly effective. Such stone layers should be built up with the earth portion of the dam. Paving or riprap is frequently placed after earthwork is wholly or nearly finished.

Concrete slabs are not as durable as riprap or rubble paving; they have the further disadvantage that the smooth surface carries waves farther up the slope. Joints must be provided for settlement and expansion, as well as for construction purposes. If the slabs rest on sand, a sill must be provided to close the joint, or wave action will wash sand through; this has occurred at Minatare dam,[26] and elsewhere. Paving at Belle Fourche[27] dam resting on

* See also p. 538.

† For costs on Encino dam, see *E. N. R.*, Apr. 30, 1925, p. 737.

24 in. of gravel, and laid dry, was displaced by waves engendered by a wind velocity of 72 mi. per hr. Relaid, after replacing the gravel, and all joints pointed. After three seasons (with temperature range between 30 and 110°F.), no cracks greater than hairline are evident.

Reinforced-concrete paving must be either jointed or reinforced to conform to settlement without loss of integrity. Lack of bond in horizontal reinforcing resulted in failure of paving on a dam near Trinidad, Colo.[4] On Henshaw dam,[17] Cal., 4-in. paving of 1:3:6 concrete was reinforced with welded mesh. On Lower Reservoir dam, Balmorhea, Tex.,[28] woven wire reinforcement was placed diagonally to the expansion joints, crossing them at the step (step detail was similar to Fig. 227). A thin felt strip laid over reinforcing on the step provided plane of separation. Strength and flexibility were required.

*Durability.** Wave action causes failure of many pavings. Water slope of Riverside reservoir was paved in 1907, with reinforced concrete ($3\frac{1}{2}$ in. mesh, No. 10 wire), 3 to 4 in. thick, slope 1 on 1.5, in 10-ft. strips, with butt joints; no vertical joints. Wave impact from 4.5-mi. stretch thrust water into the butt

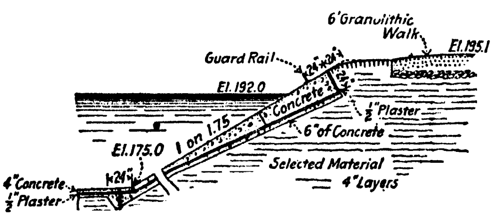

FIG. 82.—Paving, Forbes Hill reservoir, Quincy, Mass.[24]

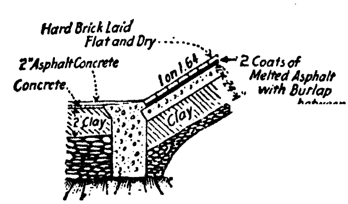

FIG. 83.—Queen Lane reservoir, Phila. Junction of bottom lining and slope paving.

joints, washed out the sand, and undermined paving, causing failure. Reinforcement poorly placed, some of it being entirely below the concrete. New paving was in blocks, 10 by 15 ft., all joints stepped; thickness, 5 in. Where new paving was placed above old, butt joints were used, breaking joint with the lower course; two layers of tarred felt were placed in the joints.[29]

Most of the paved slopes of Sudbury reservoir are 1 on 2. Paving is 18 in. thick, thoroughly bedded on 12 in. of gravel, and laid by hand; interstices filled with smaller stones, and spalls driven in. Examination 5 years after laying showed that wave action had caused uniform settlement which in no wise injured the paving. Where not exposed to wave action, paving still kept to line and grade.

Erosion of earth embankments by wave action may be greatly retarded by a floating timber boom, anchored about 3 ft. from the bank. Spring Valley Water Co., San Francisco, has for years used log booms anchored a short distance above its earth dams to still waves. Booms last about 20 years.[30]

Downstream Slope and Top. Top, upper part of water slope, and downstream slope are usually covered with soil and grassed (see p. 112) as a protection against rain, erosion, etc., and for more pleasing appearance. In regions of little rain these parts are sometimes covered with gravel and kept free from vegetation. If field stones or quarry fragments are abundant, or if many

* See also two preceding paragraphs.

cobbles are removed from the earth fill, the downstream slope is sometimes covered with them, giving a pleasing appearance. Gravel or other porous material should be placed under coarse stone to prevent erosion by surface drainage. If a stone layer is used merely for convenient disposal, or intended as a drain, and grassing is proposed, surface interstices must be filled with small stone before soil is applied. Slips of grassed water slopes on north dike of Ashokan reservoir, due to heavy rains of 1916, were stopped by rows of planks set on edge with tops 1.5 in. below grade, held by stakes driven down 3 ft. Downstream toe of Priest dam, Cal.,[20] is rock fill from tunnel spoil, faced with paving. Throttle dam,[13] Raton, N. M., has downstream paving 3 to 6 ft. deep. Twice during construction, when 66 per cent. complete, this dam was submerged by flood waters "without serious injury to embankment." Downstream toes of some dams are riprapped to a depth as great as 10 ft. to resist backwash of spillway discharge. For grassing, see p. 112.

Costs. Justin[14d] estimates slope protection to cost ½ to ¼ the total cost of embankment. To reduce costs at Deer Flat reservoir, U. S. Reclamation Service, coarse gravel was dumped on the finished 1 on 3 slopes so that the top width was increased from 20 ft. to 51 to 67 ft.; the cost was such that graveling can be repeated twice before cost of riprap paving the whole slope is equaled. During four working seasons, beaches and terraces have been formed and subsequently obliterated by working down the slope.[31] For latest surveys, see Reclamation Record, March, 1923.

DESIGN AND CONSTRUCTION*

Limitations in Design.† No mathematical formulas nor even general rules can be stated for designing earth dams. Long experience has established some dimensions approximately and some methods of construction. Intelligent selection and use of materials in design, and scrupulous, painstaking faithfulness in construction, are essential. A wide variety of earths may be used, either alone or in combinations, but nature of materials adopted must largely control design and methods of construction.

Dimensions at top should be so determined as to insure ample thickness at and below full-reservoir level. Available materials, size of reservoir, and necessary factor of safety will principally determine. Height and weight above water level are needed to resist water and ice pressures and maintain compactness to resist percolation. The depth to which frost tends to make the earth in the top of a dam friable and pervious must not be overlooked.

Top width should not be less than 10 ft., except in unimportant dams; requirements for highway may increase this to 30, 40, 50 ft. or even more. Top width of Huffman dam, Miami Conservancy District, is 30 ft. For highway details, see *E. N. R.*, Oct. 23, 1924, p. 661.

Justin[14e] quotes as rules: (*a*) 5 ft. + height ÷ 5; (*b*) height ÷ 4.

Freeboard. The top of a dam should be so high that even tops of waves will not send water over. Waves will run up paving beyond their height. If back of the dam there is a long stretch of reservoir in the direction of persistent

* For Construction Plant on Miami Conservancy District see Part X, Technical Reports, 1925.

† Justin attempts to apply formulas in "The Design of Earth Dams," *T. A. S. C. E.*, Vol. 87, 1924, p. 1.

winds, friction of wind on the water surface will raise the water level against the dam as well as making high waves. These may be combined with a flood. Height above full reservoir should not be less than 5 ft. in small reservoirs or 10 ft. for larger ones. For important dams it is frequently 15 to 20 ft. Table 58 indicates American practice.

Slopes and Berms. Stability of earthworks depends on the friction of component parts, which is measured by the angle of repose. "Friction is greatest for coarse and least for fine soils . . . A slight addition of moisture increases the coefficient of friction, but an excess of water acts as an unguent in diminishing the friction."[3d] Slopes are determined largely by the character of material* and the position it would assume when saturated, also by factor of safety desired and importance of dam. Burr Bassell[32] would make cross-sections heavy enough to afford factor of safety of 10 against sliding.

Slopes for dams not exceeding 30 to 40 ft. in height may be uniform from bottom to top—1 on $1\frac{1}{2}$ to 1 on 3 for the water side, and 1 on $1\frac{1}{2}$ to 1 on 2 for the downstream side. Slopes on the water sides of high dams should not be steeper, as an average, than 1 on $2\frac{1}{2}$. Much flatter slopes have been used, as flat as 1 on 5 for compacted embankments, and very much flatter in special structures. For high dams, horizontal berms should be used to buttress the slope at vertical intervals not exceeding 30 ft. and the slopes should be flattened toward the bottom, imitating a natural hill.

Settlement† and Shrinkage. In computing earthwork quantities, consideration must be given to the larger space occupied by freshly excavated material, and to the fact that, in a compacted embankment, the volume becomes 8 to 15 per cent. less than the original; part of this is due to waste in handling. Allowance must be made in order to secure the desired dimensions and elevations in the settled structure. Slopes are trimmed subsequently. Indian engineers add 2.5 to 3.5 per cent. to height;[3e] Justin suggests 3 to 5 per cent.[14f] Uniformity of settlement is essential; otherwise slips will occur. If dam is placed in layers too great for proper compacting, settlement will not be uniform. Method of constructing dam near Trinidad, Colo., 45 ft. high, caused settlement of 6 ft. To regain proposed height, material had to be placed on downstream toe.[4] No settlement of embankment for Hillview reservoir,‡ placed in 4-in. layers, has been detected.[33] Settlement after 3 years on two Porto Rican dams varied from 0.26 to 3 per cent.[34] No allowance for settlement was made on Standley Lake dam; it settled 12 per cent., so that top was below spillway level.[35] Settlement cracks allowed leakage to cause failure of Apishapa dam.[36]

Materials. Impermeability is desirable in an earth structure, if cohesion is to be utilized, and dangers of piping lessened.[3f] Justin[14g] claims that, notwithstanding this generally accepted requirement, among materials found in dams successfully constructed in conformity with the inherent properties of each, may be mentioned loose rock, sand (fine to coarse), gravel and silt, rock flour, soil, and clay. For very dense compacting, earth should be placed in 4-in. layers and rolled under a pressure of at least 30 lb. per sq. in., until consolidation results. The best natural materials are those which, when dry,

* Slopes at Gilboa dam were flattened to conform to the characteristics of the materials.[6]

† See also p. 214.

‡ No observations later than 1915.

break into "tough," not brittle, fragments, and have a dull and irregular fracture." At Lahontan dam, best material was found to be an artificial mixture of gravel with equal parts of soil or silt.[37] Some earths which, when thoroughly saturated would assume slopes flatter than the face of the dam, may be used to a limited extent in dams of liberal section by mixing with more stable materials or by placing in the heart of dam with a considerable thickness of better material outside. Paving adds to the safety of such construction.

Weight of Rolled Earth Embankments. Average of twelve samples, rolled in layers 4 in. thick after compacting, 136.9 lb. per cu. ft. Average of eight samples, rolled in layers 6 in. thick after compacting, 138.4 lb. per cu. ft. Samples were taken from five well-distributed places on dikes for Ashokan reservoir. The material was mostly clayey loam; some samples were sandy loam and others gumbo, all with varying percentages of small stones excepting that two gumbo samples contained no stones.

Plane of Saturation. Strange[3] says that few existing dams are impervious. Researches by Croes, Smith, and Sweet determined that in an earth dam, with masonry core, there was a continuous water plane, extending from the water surface of the reservoir to the core wall, and on the downstream side to the toe, having a maximum inclination of about 20 per cent., showing that cores are not watertight, as the dam was saturated below this plane.

The slope of this plane of saturation indicates the degree of tightness, and its height, the decrease in weight and stability of the dam. Material below this plane has less resistance to slipping. Necessary conditions to stability are: curtailment of the quantity of seepage so that no erosion can occur, and restriction of plane of saturation within the base of the embankment.

Investigations* have been made at the Croton dams and elsewhere.† U. S. Dept. of Agriculture is studying saturation of existing dams. Tests at Titicus dam,‡ by measuring height of water in pipes set in the embankment, showed that the water level in the upstream pipes fluctuated with that in the reservoir, while that in downstream pipes corresponded more closely with ground-water level, thereby indicating that the core wall formed a barrier to passage of water. From these experiments, it was concluded that shape of dam, natural or artificial underdrains, springs or leaks (other than percolation), and rainfall, controlled amount of saturation, if not entirely, fully as much as the imperviousness of the core wall and the earth embankment. A core wall cannot be depended upon to prevent saturation of the downstream slope.

Cores in dikes of Wachusett reservoir consist of 6-in. layers of fine loam soil well sprinkled and rolled. Experiments have shown that while the plane of saturation on the reservoir side was at water level, immediately below the core it dropped to a level slightly below the base of the dam. Measurements proved that the water draining out of the dike was not in excess of the natural drainage from precipitation on the area of the dike itself. (See also p. 205 and *E.R.*, Nov. 30, 1901, p. 520.)

* See bibliography, T. A. S. C. E., Vol. 87, 1924, p. 2.
† For results at Coquille dam, see T. A. S. C. E., Vol. 87, 1924, p. 124.
‡ On Croton watershed. Core walls of this dam and other Croton dams tested, are of rubble masonry in Rosendale natural cement mortar.

Justin[50g] gives various analytical studies of the location of the plane, which are of qualitative value, although premised on formulas derived from laboratory materials.

Drainage. To keep the plane of saturation as low as possible, will decrease the tendency to slough and thereby increase stability. This is accomplished by placing the more pervious material in the downstream part of the dam with an ample loose stone drain under it, and a collecting pipe beneath. From the latter, a pipe with tight joints is laid to a safe point of discharge; and arrangements are commonly made for observing or measuring the flow, so that if increase occurs, reasons may be promptly sought. If seepage is found slightly muddy, it indicates that fine material is being washed out of the dam. This means that a water passage is being formed which is likely to enlarge, perhaps rapidly, and cause trouble or disaster.

Downstream slopes should be protected from erosion by preventing concentration of surface drainage. Gutters should be made along the inner sides of berms, and water thus collected taken into a pipe drain laid beneath. In a long dam, pipe drains should be laid from the berm drains at suitable intervals down the slope, and the drainage led to a safe point of disposal. Manholes should be built on pipe drains at convenient points for inspection and cleaning. On Sherburne Lakes dam, Montana, ground-water conditions in the flanking hills required an unusually complete system of drainage, involving a core of screened gravel, the base of which was drained by outlet pipes.[38]

Foundations for earth dams should be firm and impermeable, of a character to support the load without sensible compression, and free from fissures and porous layers. Strange gives the rule:

> All soils which are suitable for the formation of the dam are also good for its foundation: those which are unsuitable for making the embankment are bad for its foundation and should be removed.[3h]

If their extent makes this impracticable, some form of cut-off must be employed, or site rejected. Springs should be led outside the site. Seepage should be minimized by clearing, grubbing, and, where necessary, furrowing the dam site, and removing organic or porous material. Foundation seepage is held accountable for some failures, notably at Horse Creek dam.[39]

Outlets* and stream control conduits may be separate structures, or one fulfilling both functions. Unprotected pipes should not be laid through embankments below water level; unequal settlements, even if small, may cause breaks. Water is likely to follow along outside of any conduit. If rock is near surface, or dam is on very compact earth, masonry outlet may pass beneath embankment, with frequent cut-off walls to prevent flow of water along outside.† Great care must be exercised in compacting embankment to get tight contact with conduit at every point to prevent seepage. In some situations, as at Henshaw dam,[17] a tunnel around end of dam is best outlet; in others, outlet may be distant from dam, either tunnel or conduit in trench through natural ridge. For small works, siphon pipe over dam may be satisfactory outlet, if means for removing air at summit are provided.‡

* See also p. 105.
† Many failures are laid to this omission, notably Sevier River, Utah.[40]
‡ See p. 779.

*Stream Control.** Stream control problems have been eliminated on some western dams by completion of whole structure between consecutive rainy seasons; at Henshaw dam 400,000 cu. yd. were placed in one season.[17] Double-barreled conduits, as at Sherburne Lakes[38] and Wanaque,[11] facilitate handling stream during construction. At Wanaque, the first conduit is 3 ft. higher than the second, and was built above water level in a dry spell; the lower conduit, at stream-bed level, was constructed after a temporary dam had deflected water through the high-level barrel. When a conduit for stream control only is no longer needed, it should be carefully filled with masonry wholly or for a sufficient portion of its length, before water attains much depth in reservoir.

Outlets should be controlled by a *gate-house.** If outlet is through dam, gate-house may be in reservoir near heel of dam, or, if the dam has a substantial masonry core wall, the inlet gate-house may be built integral with it. Either method affords security and convenience at a lower cost than a separate outlet. Water is led to the second type of gate-house by wing walls retaining the embankment, or by an extension of the culvert. Outlet ports should be at different depths to take off water according to quality. On large works a second gate-house, termed "lower gate-house," is generally located at downstream toe of dam, and contains valves for regulating and controlling flow into aqueduct. Pipes in conduit generally connect the two gate-houses.

Construction work should be scheduled to avoid the rainy seasons, as excessively wet fill cannot be rolled. Some moisture, however, is required for proper compacting, and if the material be delivered too dry, it must be sprinkled. Methods of construction should be such as to secure a dense, uniform mass, to minimize seepage, shrinkage, and settlement, and so placed in horizontal layers that any cleavage planes that might accidently occur will not promote slipping. Bond between layers should be secured by scarifying and sprinkling, if necessary.

Table 53. Pressures per Linear Inch of Wheel Width Developed by Road Rollers†

Makers	Numbers of wheels	Nominal tonnage	Weight in lb. on wheel carrying max. load	Pressures, lb. per lin. in. of wheel width
Austin Motor	3	7	....	359
Buffalo Springfield	2	8	10,050	250
Austin Motor	2	8	....	257
Austin Motor	3	8	....	334‡
Buffalo Pitts	3	10	8,170	454
Austin Motor	3	10	....	418
Monarch	3	10	7,400	370
Buffalo Springfield	2	10	13,330	250
Kelly-Springfield	3	10	7,800	390
Austin Motor	3	12	....	455
Buffalo Pitts	3	12	9,170	460
Kelly Springfield	3	13	9,800	445
Austin Motor	3	15	....	450‡
Buffalo Pitts	3	15	11,830	535

* See also Chap. 6, p. 205.
† Information furnished by Buffalo-Springfield Roller Co., Springfield, Ohio, and Austin Western Road Machinery Co., Chicago.
‡ Wider rolls than next size lower of same make.

Sequence of Construction. Construction should begin at the bottom of the valley and can be carried up substantially level. If a temporary gap must be left for stream, railroad, or highway, great care must be exercised when filling it to make impervious junctions. Sides of gaps ought not to be very steep and should be trimmed free of poorly compacted material just in advance of placing earth in the gap. Allow dam to stand dry as long as possible, before subjecting to saturation. When a dam built dry is subjected to water pressure, equilibrium must be reestablished internally. A few western dams have shown increased leakage after a drought when the water level is raised.[14j]

Placing. Earth may be brought on to the dam by wagons, cars, cableways, or baskets carried by men (for sluicing, see p. 216). At Lahontan dam, 700,000 cu. yd. were delivered by belt conveyor, 925 ft. long.[37] Tractors, or wagon loaders pulled by tractors, will prove economical at borrow pits. Earth should be deposited systematically, spread to layers of uniform thickness by scrapers or by hand, and stones of larger diam. than the thickness of the layer after compacting removed. Even much smaller stones cannot be permitted in nests or as a large proportion of the material. If selection or mixing of earth is necessary, it can be accomplished by proper placing of loads as delivered; harrowing may sometimes be advantageous.

Compacting during construction is essential to consolidation; it is commonly done with power or horse rollers* having loose rings, grooves, cleats, or other devices for concentrating the loads. Occasionally specifications stipulate a pressure of 30 lb. per sq. in.; the more usual specification, and the one preferred by the roller manufacturers, stipulates the pressure per linear inch of tread of the larger wheel. On Keechelus dam, U. S. Reclamation Service[8] required: "6 impressions with a 12-ton traction engine giving about 300 lb. pressure per in. of tread of rear wheels." The necessary number of passes of the roller depends upon its weight, nature, material to be compacted, and several other circumstances, and can be best determined by trial. The mistake is often made of using thicker layers and a heavier roller. Some layers of the failed Horse Creek dam were 4 ft. thick; no rolling.[39] Droves of horses driven back and forth are effective. If the earth is brought in wagons, much of the compacting can be secured by judiciously distributing the travel. On Bassano dam,[41] spreading by 4-horse Fresno scrapers to a thin layer resulted in such incidental compacting that the clause in specifications requiring rolling was not enforced. Since neither horse nor power rollers can compact the earth close against the wall, this portion of the embankment on each side must be very carefully rammed by hand or very thoroughly puddled so as to be homogeneous with adjoining parts of the bank and in tight contact with the wall.† In general the upstream half of the dam (*i.e.*, the portion intended to be most impermeable), should be so compact that, after removing the looser surface material at any stage of work, a pick, crowbar or shovel cannot be driven into the bank more than an inch or two by one strong stroke. Thickness of earth layers in the most impermeable part of a dam is commonly 3, 4, 5, or 6 in. after compacting, depending upon character of materials, degree of security desired, permissible expenditure, and judgment of the engineer. In remaining earth

* Dumping from a trestle never constitutes proper consolidation.

† On some dikes of Ashokan reservoir special 19-ton rollers had rear wheels placed so as to compact the earth close to the core wall.

portions, layers are often of considerably greater thickness and less thoroughly compacted. Nowhere may work be so done that any serious settlement or slip can occur if the material becomes wholly or nearly saturated, as is frequently the case, due to seepage or rain or both.

Hydraulic Consolidation. Dry earth on dam at Wichita Falls, Tex.,[42] was placed in fills 3 ft. deep, and flooded; layers consolidated and dried out ready for the succeeding lift in 7 to 12 days. Material was favorable to process, contained 45 per cent. voids, and saturated and dried out readily. On dams with heavy soils, the experience of Morgan Engineering Co.[42] has been that layers are limited to 12 or 18 in. This "irrigation" or "ridge and wet-trench" method was also used on Balmorhea project,[28] and Costilla Creek, N. M.[43]

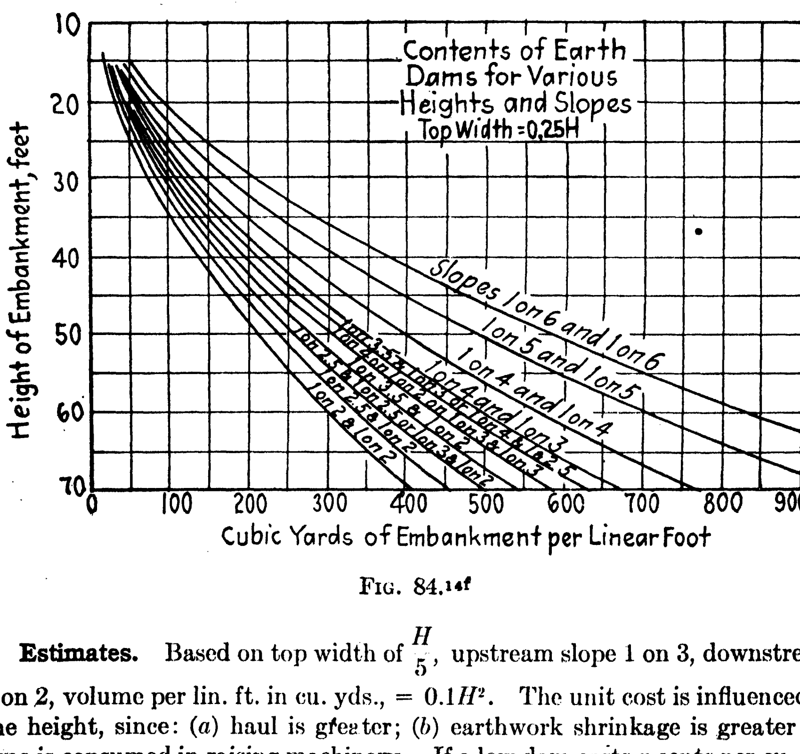

FIG. 84.[14f]

Estimates. Based on top width of $\frac{H}{5}$, upstream slope 1 on 3, downstream, 1 on 2, volume per lin. ft. in cu. yds., $= 0.1H^2$. The unit cost is influenced by the height, since: (*a*) haul is greater; (*b*) earthwork shrinkage is greater; (*c*) time is consumed in raising machinery. If a low dam costs x cents per cu. yd., cost in cents per lin. ft. for one H ft. high is given by Johnston[44] as $0.1H^2\left(x + \frac{2H}{30}\right)$.

Damage to Earth Structures by Burrowing Animals. Of 61 recorded failures, most are blamed to burrowing animals, especially when other plausible reasons are lacking. Muskrats worked such havoc to embankments of the Delaware and Raritan Canal that employees received a bonus for each one killed. Animals generally attack unprotected banks, but instances are on record in India of mortar being picked out of interstices in stone paving. Animals found in many regions liable to injure earthworks are: rat, mouse, weasel, common hare, rabbit, common porcupine, badger, earthworm, locust, eel, turtle, snake,

crawfish, kingfisher, fox, skunk, beaver, woodchuck, mink, mole or shrew, and muskrat. The muskrat is undoubtedly the most dangerous. Animals actually known to have injured earthworks are: muskrat, woodchuck, Indian rat, yabba, snake (in India), fiddler crab, gopher, kingfisher, mole, mollusk, eel, crawfish, mouse, and beaver. The various ways of preventing inroads of burrowing animals are: (1) resistance of hard-rolled homogeneous material; (2) core wall of puddle, stone, concrete or timber; (3) paving on upstream or both slopes; (4) sodding downstream slope; (5) poison has been placed in the water of some navigation canals, but naturally this is not applicable to water supplies; (6) employment of men to patrol embankments. (See also p. 196.)

Failures. Especially in structures insusceptible of mathematical analysis, failures offer important guides to successful construction. Of 63 earth dam failures recorded by Justin,[14k] 11 were due to piping, slips, and sloughing; 27 to inadequate spillways; 11 to poor construction along outlet; 3 to internal pressure; 4 to foundation weaknesses; 2 to unknown; 1 to burrowing animals; and 4 to miscellaneous causes.* Increased ground-water head has been known to force water under foundations. It may saturate the abutting banks of the valley and cause sloughing. Earth dams which have failed have been mostly minor structures, built without engineering supervision, or built in disregard of engineering advice. Even that most disastrous break, to which allusion is still so frequently made, the Johnstown flood, was caused by disregard of simplest principles of earth dam construction, such as could not possibly occur under direction of a fit waterworks engineer.

HYDRAULIC FILL DAMS

Characteristics. Hydraulic fill dams are built by selective deposition of earthy materials from flowing water. Requirements of top width, height, slope protection, and berms are similar to those for dams built dry. The fine material deposited in the core furnishes the watertight element. Most economical conditions are suitable materials at each end of dam site above level of top of dam, and abundant water under sufficient head for hydraulic excavation and sluicing.

Methods of Construction. If the entire process is hydraulic, as is the general western practice, materials are excavated by nozzle jets, transported in sluices to the dam site, and selectively deposited by control of velocity. The heavier materials drop near outlet, and the fines ultimately settle in the core pool, the basin for which is formed by prior construction of low dikes along the edges of the dam at successive stages. When deposited, the core is fluid and is held in position by the outer portions of coarse materials. Where grades for transportation cannot be obtained, hydraulic selection may be secured: (1) by dumping all materials dry on the edges, whence nozzles wash the fines into the core,† as at Somerset, Vt.;[45] or (2) by dumping materials into boxes, where nozzles break it up and wash it into a pump-sump protected by a "grizzly," whence it is pumped to place of final disposition, as at Miami, Ohio.[46] Both methods† are termed "semihydraulic" (see p. 215).

* See also "Record of 100 Failures," by Jorgensen, *J. Elec.*, Mar. 15 and Apr. 1, 1920.
† Dams built by this method include: Bridgewater, N. C.;[45] Davis Bridge, Vt.;[47] Henshaw, Cal.;[17] Somerset, Vt.;[45] and Willow River, Ore.

Recent hydraulic-fill dams include those of the Miami project,[45] Terrace,[48] Santa Maria, and Onion Valley.*

Advantages are: (1) Rainy weather† will not halt work. (2) Power is largely substituted for muscle. (3) In a narrow gorge, as at Henshaw dam,[17] construction is quicker than by dry methods, due to interference of teams. (4) Due to extra pressures exerted by the core until drained, the greatest stresses occur during construction, thus affording a factor of safety in the completed dam. (5) Cost is less. For costs at Terrace reservoir dam, see *E. N. R.*, Apr. 7, 1921, p. 599. Hazen says:

> The advantages and economy of the hydraulic method are such that it will often be good business to use it, even though the required volume of the dam would be greater. [50a]

Disadvantages. For a successful dam, rigid control must be had at all stages of construction. There have been conspicuous failures at Necaxa[51] and Calaveras,[52] which have tended towards its disrepute. If the materials available do not contain the proper proportions of coarse and fines, nor the right sizes, a satisfactory dam cannot result. O'Shaughnessy[50b] doubts the economy in this type; Calaveras construction costs were 45 cts. per cu. yd. although the estimate was 20 cts. Elliott[50c] warns against choice of this type on the sole ground of economy, when some other type might better meet conditions. With fluid cores and inadequate slopes, an instability exists which was not recognized in the earlier designs.

Desiderata for good design and safe construction are: (1) sufficient coarse, heavy material in the slopes, segregated by flowing water in such manner as to form a progressively denser mass inward; (2) care to avoid introduction of any fines, to act as a lubricant in the mass of the slopes; (3) enough granular material in the core to minimize its plasticity, and enough clay to secure substantial imperviousness.

Materials above solid rock are most easily worked and have been most extensively used. Most suitable material is an admixture of soil, clay, sand, gravel of all sizes, and rounded boulders up to the maximum sizes which can be transported with the water and grades of sluices, ditches, or pipes, available. For good results, clay should constitute 10 to 30 per cent. of the whole; it acts as a lubricant during transportation as well as rendering the dam tight. Angular stones may be used, especially if the proportion of clay is large, but they are much more liable to lodge in sluices, cause greater wear, and maximum sizes which can be transported are smaller. Pure clay is the most difficult material to use, because most unstable while saturated with the water needed to transport it and because it gives up water slowly: shrinkage is much greater, cracks are more likely to open during maturing process (drying out and settling), and it is more difficult to maintain slopes. Nevertheless, "wet clay under pressure will part with its water even though the mass be entirely surrounded by that liquid," and when finally consolidated, a dam of this material cannot be excelled for watertightness and solidity. Clay with finid below a certain minimum eliminated makes good core, and when overlaes with sand, gravel, and rock in outer portions, its surplus water is squeezed

* See also "Reservoirs for Irrigation, etc.," J. D. Schuyler (Wiley, 1908).
† Junction dam, Mich., was built during winter months.[49]

out and necessary drainage afforded. Almost any kind of clay can be used in this way, but surface clays and soils thoroughly weathered naturally are preferable to deep-seated clays in beds of such depths as to have undergone few changes by the elements. Fine sand, glacial flour, rock dust, or any finely pulverized non-plastic material is preferable to pure clay, for the reason that when settled in water it is not subject to shrinkage or further settlement, and is practically impervious if the particles are fine enough to pass a 100-mesh-per-in. sieve. Mixtures of fine sand and clay are best of all. The suitability of materials is often determined by weighing. Dams built by Creager on Beaver River are of a "truly sand" material, the proportion of particles over ¼ in. being practically negligible. The fines are also sand.[64] At Terrace reservoir dam, the terminal moraine deposits contained 15 to 25 per cent. of rock, 0.5 to 3 cu. ft. in volume, with occasional masses of rock in excess of 1 cu. yd.[48] At Calaveras dam,[1] rock intercepted by the grizzly was passed through a crusher.

Analysis, at borrow pits for Henshaw dam,[17] of material made up of 66 per cent. sand and gravel and 33 per cent. clay are given in Table 54.

Table 54

Mesh per inch	Per cent.
200	22.30
100	39.50
80	52.50
50	66.00
40	72.80
30	80.00
20	89.60
10	98.45
Material washed out, per cent	34.25
Per cent. material passing after washing	20.75
Total fines washed out and passing 100 mesh	55.00

Similar analyses for Somerset dam are reported by Holmes in *E. N.*, June 4, 1914, p. 1237. On Tieton dam, the material placed next to core was sized from 1¼ in. to 80 mesh.[15]

Unsuitable material has been held responsible for several failures of hydraulic fill dams; at Schaefer dam[50v] the material was either yellow clay, or rough, angular sand rock, with very little sand or loam; the clay puddle core remained the consistence of cream.

Core is the most important element. It should consist of the last particles to settle out of the water of transportation, the "fines." Early practice used a core width two-thirds that of the dam. Recent practice favors a narrower core. Elliott[50c] and others favor a core pond width not more than 20 per cent. of the total thickness of the dam at all elevations. Advantages are: (1) It is wide enough for facility of construction. (2) It is sufficient for watertightness. (3) There is no danger of tongues of coarse material encroaching on the core. The practice of Miami Conservancy District[50d] was to limit width of core to height of dam above the given level; this resulted in a maximum width of core of 20 per cent. of thickness of dam. There is little overhanging of the

coarse material, and under such conditions are eliminated: (1) sloughing of coarser material into core; (2) squeezing of fines into slope; (3) bursting pressures. Burr[50e] also favors a reduction in core width. Hazen[50j] recommends that core be made as narrow as possible; critics question advisability of this, especially since under the stress of construction, it is not practicable to control the core lines to a nicety. There is always a danger of the slope material sloughing across the core and forming a tongue of pervious material through the dam.

Depth of core pond should not be too great, or hydrostatic pressures on slopes will be excessive. A depth of 1.5 to 2 ft. was used at Quemahoning dam.[50f]

*Solidification** of the core from a fluid into a stable element is accomplished through drainage† and compacting. Opinions differ as to the more important agency. Hazen argues that water content under pressure creates a quicksand condition:

> The puddle clay core of an hydraulic-fill dam is physically like quicksand, but with particles one hundred times smaller in diameter and a million times smaller in weight. It has the instability of quicksand in full measure and it retains it for a long time, or perhaps indefinitely.[50g]

Fines. Some cores are of too fine material and the retained water hinders solidification. The proper size for core material is not yet established; Hazen[50b] concludes:

1. It is not well to build an hydraulic-fill dam of material of which any large percentage consists of clay or of particles less than 0.01 mm.‡ in diam.; and in general all such smaller particles may well be wasted and excluded from the dam.

2. By reducing the construction pool to a minimum, and by controlling it and the quantities of water used for sluicing, the core material may be held to a certain degree of coarseness by wasting all smaller particles. An effective size of 0.01 mm. may reasonably be sought. Elliott[50c] argues that maintenance of proper relation between slope and core will automatically take care of grain size. At Henshaw dam[17] about 20 per cent. of fines were wasted, and used to seal reservoir bottom. Several notable failures of hydraulic-fill dams indicate the danger of rapidly building a dam with a large per cent. of very fine material. Paul[50j] doubts practicability of maintaining a limit of fineness of 0.01 mm.; cost of construction of Miami Conservancy dams would have been greatly increased, and presumably imperviousness of cores lessened. Satisfactory consolidation in cores of these dams is taking place, despite the fact that about 40 to 60 per cent. of the material is finer than 0.01 mm. On the Conservancy and other dams excess of fine material was controlled by mechanical analysis.

Muckleston[50k] cites a sandy clay soil placed behind a retaining wall hydraulically, which never consolidated and after a year was removed by discharging through 1-in. holes drilled in the wall; the material poured out much like thick molasses.

* See also "Settlement," p. 214.
† Hollow core wall to promote drainage is patented (No. 1011427).
‡ Henny's description, *T. A. S. C. E.*, Vol. 74, 1911, p. 82, is endorsed by Hazen: "fine sandy silt of the arid West, having little cohesiveness, good self-drainage properties, becoming hard, and solid after a short time."

Table 55. Typical Analyses of Core Samples
Miami Conservancy Dams [50j]

	Bureau of Soils, standard sizing, in millimeters	Germantown dam, percentage	Englewood dam, percentage	Lockington dam, percentage	Taylorsville dam, percentage	Huffman dam, percentage
Fine gravel	2 to 1	0	0	0	0	0
Coarse sand	1 to 0.5	0	0.4	0	0	0
Medium sand	0.5 to 0.25	0	0.3	0	0.1	0
Fine sand	0.25 to 0.10	4.2	4.6	1.0	1.9	2.8
Very fine sand	0.10 to 0.05	17.3	19.6	10.1	18.1	15.0
Silt	0.05 to 0.005	56.0	52.7	59.6	59.7	52.5
Clay	0.005 to 0	22.5	22.6	29.3	20.2	29.8

Table 56. Effective Size and Uniformity Coefficient of Core Materials
Holmes[50l]

	Catawba	Linville	Paddy Creek
Size at 60 per cent. by weight, in millimeters	0.146	0.150	0.157
÷ Effective size (at 10 per cent. by weight), in millimeters	0.026	0.028	0.030
= Uniformity coefficient	5.6	5.4	5.2

Slopes. During construction, low dikes on either slope are carried up in advance to retain the core pool. In a theoretical section with core width decreasing toward top, the coarse material partly overlies the core, and in the completed compacted dam, the core must support the slopes. The slopes of the core are an important element in stability of the dam; if too flat, the superimposed weight of the granular material of the slopes will be too great

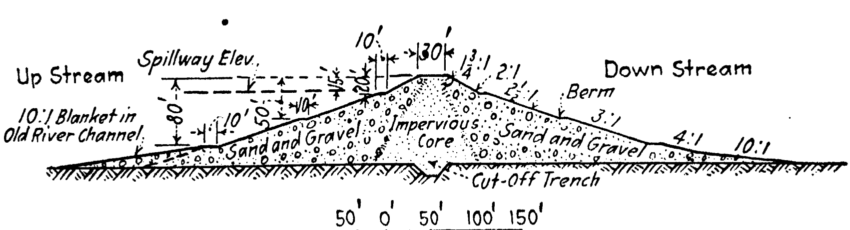

FIG. 85.—Miami Conservancy District dams. (T. A. S. C. E., Vol. 85, 1922, p. 1509.)

for the uncompacted core to sustain, and some of the coarse material will slough into the core, lessening its imperviousness. Sloughing was detected by rod soundings on Miami Conservancy dams.[53]

Slope material should constitute two-thirds to four-fifths of the bulk, equally divided between upstream and downstream sides; it should be heavy and free draining (small boulders, stone fragments, gravel, etc.).

Rock fill may be used for downstream portion and hydraulic fill upstream, if local conditions so dictate. If sluiced materials are deficient in rock, gravel, and coarse sand, brush is used in maintaining the edge dikes, banks, or levees. Slopes constructed by the hydraulic process have the advantage of being

freed from slimes, and it has been found that they are better packed than those dumped in place.

Schuyler attempted to secure a gradation of material from coarse at the faces to fine in the center. This would insure greater voids at the faces and better drainage, and would reduce damage caused by seepage. There exists between core and slope a transitional zone of fine sand, silt, and other material readily moved by water, but which will settle several feet in an hour. Morgan[50m] found that this zone had a variable width; it was assumed 25 ft. wide on each side of the core in the Miami Conservancy dams.

Limits. Hazen[50o] computes the slope required on the basis of a coefficient of friction of 0.5 and equal weights for outer and core materials, and finds that a factor of safety of 3.0 requires a slope of 1 on 6. He recommends:[50p] Make the outer portions large enough safely to resist the pressure of the core as a liquid until there is satisfactory evidence that solidification of the core has so progressed that horizontal pressures are negligible. He would further stabilize the outer portions by so placing rock fill on them as to minimize voids, and reduce the chance of settlement. Slopes should be sufficiently flat to insure the stability of the dam under all conditions of saturation. In most cases a slope of 1 on 3 upstream and 1 on 2 downstream will insure this, if the correct materials are used and if they are correctly placed.[50q]

Foundations must be naturally stable and impermeable under expected reservoir pressure, or must be made so. This is particularly important where the reservoir is situated on porous lava flows as in northwestern United States. The wastage of the fines from the core assists in silting the reservoir bottom.* A dam on Bridgewater project,[54] N. C., was founded on gneiss, composed of saw-tooth strata giving a good bond for the earth-fill materials. Foundations should be prepared as for other classes of earth dams, including cutoffs. Concrete walls, steel or wooden piling extending down into impervious material, are sometimes carried up for greater or less heights into the dam, the piling being footed in concrete.

Settlement. Since, during construction and maturing, parts of hydraulic-fill dams change from liquid mud in unstable equilibrium to solid bank, considerable settlement takes place and must be provided for. This process is sometimes very slow, and in a large dam may limit rate of construction. It is good practice to test dam at all stages by permitting water to rise in reservoir to within 10 or 15 ft. of top. Hazen[50s] concludes that full stability may be reached when the voids in the core material have reached between 50 and 40 per cent., and that material with 35 per cent. voids would give greatest strength and stability. C. E. Curtis[50i] suggests the probability that the dry material placed by hand on top of Quemahoning dam settled into the semi-liquid layer of core, absorbing the water it contained, so that the whole mass became stable.

Provisions during Construction. Quemahoning dam was built to excess height of 3 ft.; observations showed an average settlement of not more than 9 in. (less than 1 per cent. of height), after 9 months.[50i] For the dam at Bridgewater, N. C.[54] contractor was required to place 10 per cent. extra material to allow for shrinkage.

* Utilized at Henshaw dam.[17]

Slides have occurred during the construction of many hydraulic dams, due to badly drained core material. Dillman[50r] says that drainage will avoid such accidents, and will cure slides. For Calaveras slides, see reference 52, p. 221.

Tests with Goldbeck pressure cells[46] demonstrate that material in core acts as a perfect fluid during early stages of settlement, until solidification ensues. The degree of solidification is generally measured by the depth to which a pipe or ball can be put down. On Quemahoning dam, Curtis[50i] found that two men could push a 1-in. pipe down 30 to 35 ft. and could drive it 60 ft. For records of penetration at Bridgewater, see *E. N. R.*, Aug. 12, 1920, p. 308. Penetration tests with a 6-in. cast-iron ball were made regularly at all Miami Conservancy dams[50j] and were considered valuable as indicating quantitatively the degree of consolidation. Investigations also made at these dams with Goldbeck[46] pressure cells indicate that the core at lower elevations is consolidating satisfactorily; the consolidation begins within a few weeks, and after a few months lateral pressures are about one-half of the vertical.

Samples are sometimes taken from soundings although their representativeness is questionable. Neither will well-drill explorations yield reliable data, as grinding results from the process. At Henshaw dam,[17] test pipes driven up from culvert (with static head on clay and gravels, of 21 lb., or 50 ft.), recorded a water pressure in the clays of 5 lb. and in gravels, 9 lb. At this pressure, a 2-in. pipe would exude 0.13 cu. ft. of clay per hr. Under these conditions, one man could work a $1\frac{1}{4}$-in. pipe 7 ft. into clay; 3 months later the gage recorded no clay pressure, no exudation took place in 12 hr., and a $\frac{9}{16}$-in. sharpened bar could be pushed into clay but 5.5 ft.*

Excess Material. At Taylorsville dam[55] for the Miami Conservancy District, measurements in fill showed an increase of 25 to 50 per cent. over measurements in cut. Material in the cut was exceedingly dense and when broken up and redeposited with coarse sizes at edges and fines in center, occupied a greater volume. At Henshaw dam,[17] due to separation of the clay from the voids in sand, material increased in volume 16 per cent.

Construction Methods. Flume or pipe for conveying material is often supported on high, wooden trestle near the center line of dam. For large dams two may be used. Laterals are provided at convenient intervals. For small dams, pipe or sluice is laid along edge of bank and raised with it, outlets being made at desired points by opening joints or by other simple means.

Stratification, *i.e.*, formation of continuous layers of pervious material through dam, is one of most important things to be guarded against. This may be controlled by even distribution of material along slopes, avoiding formation of high cones, extending beyond safe limits toward core, and by keeping pond on dam as high as possible at all times. Sand and gravel with lubricant of clay will take natural slope of between 5 and 8 per cent., but on passing into quiet pool will deposit at slope of about 1 on 1, while clay spreads through water practically level. Stratification may be corrected by systematically pushing down 1 × 12-in. wedge-shaped plank paddles to depths of about 10 ft. along lines 2 ft. apart parallel to center line of dam for a width of 20 ft. upstream from center; men work from boats or rafts and repeat operation from time to time as dam is built up. Central wooden diaphragms or parti-

* For observations of vertical and horizontal movements, see *E. N. R.*, Aug. 30, 1923, p. 343.

tions are also used to prevent stratification. Sheer boards[55] were used on Miami Conservancy dams to prevent formation of tongues of coarse material at outlet to sluice pipe; use on dam for Ochoco Irrigation District[56] to retain core temporarily, resulted in economy, control of material, and reduced danger of sloughing and settlement. A modification of the semihydraulic process at Taylorville dam consisted in saturating the loose, dry fill through drill holes until it became plastic enough to move down the slope. Fresh material was continually superposed. The cost proved to be one-half of rolled fill.[57]

Sluicing. Hydraulic excavating requires pressures of 25 to 250 or 300 lb. per sq. in. "Sluicing head" should be 5 to 20 cfs., although less may be used; 20 to 30 cfs. may be handled in one head, and is more effective proportionately than less. Records are generally compared on ratio of materials transported to water used. At Calaveras,[1] it averaged 8 per cent. Sluicing records at Ochoco dam are given in *E. N. R.*, Nov. 11, 1920, p. 940.

Nozzle of best form is the hydraulic monitor or "giant" with swivel or ball joint and interior guide vanes. Sizes vary from 2 to 10 in. in diam. At Miami,[58] 4-in. nozzle, 90-100 lb. pressure, met best the conditions *i.e.* mixed gravel and clay. Deep layers were easiest handled by blasting. At Calaveras dam,[1] jets were directed against toe of bank to undermine it; in falling, it broke up.

Grizzlies are coarse screens, to keep large objects out of pipe lines or pumps. Rotary screens at outlet to boxes at Miami[55] were found better than inclined, fixed, or traveling grates. Across the sluicing channel, and above the mud pumps at Calaveras,[1] 2-in. vertical pipes were driven 4 in. center to center screening out rock larger than 4 in.

Pumping equipment at Quemahoning dam[50i] was sufficient to deliver 11,000 g.p.m. against 250 ft. head, or 5,500 g.p.m. against 500 ft. head. At Bridgewater, N. C.,[54] to sluice core from fill on edge of slopes three-stage centrifugal pumps driven by 300-hp. motors, furnished 2000 g.p.m with a nozzle pressure of 175 lb. Pumps on a winter job at Junction dam, Mich.,[49] were housed in; no important freezing troubles. Use of manganese steel in pump shells at Miami,[57] reduced the renewal costs 90 per cent. Booster pumps are inserted on long lines. At Calaveras dam,[1] 12-in. mud pumps driven by 300-hp. motor, forced mixed material against 80-ft. head through 1000 to 4500 ft. of 14-in. pipe. Troubles with impellers led to use of manganese steel, with economy.

Sluices. With sluice grades of 6 to 10 per cent., 3000 to 8000 cu. yd. of solids can be transported in 24 hr. by a sluicing head of 10 cfs. On various dams percentage of solids transported by water has varied from 5 to about 50. Angular stones not exceeding 2000 lb. and somewhat larger rounded boulders can be carried through sluices if sufficient clay or sand is present. Sharp sand does not flow as well as rounded sand. At Quemahoning dam[50i] wooden sluices were 24 by 18 in. deep, laid on 6 per cent. grade. A depth of 6 to 8 in. would transport 500-lb. stones. Six per cent. grade was required with wooden sluices at San Pablo,[50t] and 4 per cent. when steel lined. On 2.9 per cent. grade, 200-lb. rock was carried at Terrace reservoir.[48] At Athens, Ga.,[58] required slope of 3 per cent. was maintained as dam height increased, by procuring materials

at higher levels. Schedule of operations should consider this element. To protect against rapid wear, wooden flumes may have bottoms covered with steel plates or wooden blocks set on end of grain. Most successful pipe for dam building and similar operations is one having invert of interlocking wood blocks set on end of grain, and readily removable for repairs or replacement. In tests of cast-iron *vs.* steel *vs.* spruce blocks at Terrace reservoir dam,[48] on 2.9 per cent. grade, the last proved cheapest, and as good as steel. Semicircular flumes of No. 10 gage, 30-in. diam., carried on trestles on grade of 6 to 8 per cent., were used on a winter job at Junction dam, Mich.[49] Joints consisted merely of laps; there was trouble from grade disruptions by ice in joints. Velocity of 12 ft. per sec. was used at Calaveras.[1] Greatest wear on circular pipes occurs on bottom third. Life of sluices is increased by revolving pipe (twice) after bottom wears. Welded pipe of high-carbon steel gave four times the service of spiral-riveted dredge pipe.[59] The outlet pipe at Miami[57] had several Y outlets, (known as "window pipe"), through which the finer material dropped. Canvas pipe was used on outlet at Taylorsville, to allow shifting quickly. Studies of friction heads at Miami indicated 40 ft. per 1000 as fair allowance for 12 cfs. in 15-in. pipe. Excess clay reduces friction. At Calaveras,[1] one pit of half the clay content of another required use of booster pump, rough tests on 14-in. pipe, carrying 8 cfs. indicated loss of 70 ft. per thousand, as against 15 ft. with clean water. On Piute dam,[60] wooden sluices were lined with ¼ in. hard steel; fastened by special nails with rounded heads. Time was saved on a western dam by having wooden outlet on inclined skids or trestle bents so that outlet could be readily shifted transversely of dam.[61] At Calaveras,[1] first discharge was at point farthest from pit. Removal of successive lengths changed outlet—generally without interrupting pumping.

Drainage requirements are receiving increased emphasis. Studies at Miami[53] indicate that drainage of core is due principally to water being forced upward by displacement; little escapes laterally. Water may be drained off by standpipes carried up in the dam, a ring a few inches high being added as needed, or by troughs or perforated pipes, discharging downstream or into the reservoir as may be desired. By maintaining suitable slight grades pond water may be drained toward one end of the dam or a central point for removal. At Calaveras[1] during first season, 16-in. pipe set into 18-in. pipe for protection, led to the culvert. Abandoned after core movements had deflected it and a paved channel down slopes was substituted.

Drainage of downstream portion is important in this as in other classes of earth dams. Unless approximately outer third consists of porous material, drains should be provided, extending from toe under dam, but not farther than through outer third of base. Such drains should be like filters, grading from fine to coarse materials so as to prevent water entering them from carrying fine earth out of embankment. Covered collecting well at head of each drain can be arranged to force water to rise against a slight head; each well should be surrounded with fine gravel and sand. On Paradise dam, drainage through slopes was of such proportions as to be alarming to one not familiar with the hydraulic method.[1] Hazen[50a] holds that fines seal the core and retard drainage; Morgan[50m] cites case in Minnesota where they were not a prominent factor. Dillman[50r] points out that drainage may be assured by:

(1) discard of fines; (2) retarded rate of construction; (3) narrow core; (4) insertion of tile drains. He prefers the last. Open joints of the tile pipe in Junction dam were kept free of sand by placing marsh hay over them.[49]

STATISTICS

Statistics of successful earth dams in Table 58 were compiled chiefly from Justin's table (*T. A. S. C. E.*, Vol. 87, 1924, p. 59). The dams are grouped to bring together those having the same slope at top of upstream face. Downstream slopes are expressed in terms of relation to horizontal: 2½ signifies slope is 1 on 2½. Where two slopes are given, *e.g.* (Owl Creek, 2 and 5,) the upper part of the dam has the first named slope. The upstream slopes in the "Remarks" column apply below the break, where there is a break in the slope.

Table 57. Dimensions of a Number of Earth Dams That Have Stood Successfully. Hazen[50v]

Dam	Height to flow line, H	Area of section, in square feet, A	Ratio, $\frac{A}{H^2}$	Freeboard	Width, in feet				Bottom, W	Ratio, $\frac{W}{H}$
					Top	At flow line	50 ft. below flow line	100 ft. below flow line		
* Gatun	78	99,200	16.30	30	100	397	1,469	....	1,990	25.5
* Big Meadows	64	24,300	5.93	20	30	160	460	....	548	8.56
* Coquitlam	83	38,700	5.61	15	40	145	521	....	702	8.48
* Cambria	82	31,200	4.65	13	20	98	429	....	685	8.34
Ashokan	90	35,400	4.37	20	34	112	400	....	650	7.23
Druid Lake	82	27,900	4.15	5	60	90	390	....	582	7.10
San Andreas	80	26,200	4.10	5	30	58	387	....	606	7.58
* Paddy Creek	120	58,200	4.05	25	23	161	414	664	757	6.30
* Somerset	92	33,200	3.94	13	25	78	375	....	616	6.67
San Leandro	65	16,350	3.88	5	30	56	335	....	457	7.04
* Haiwee	75	21,600	3.85	14	20	90	340	....	465	6.20
Croton	96	33,400	3.62	24	30	126	336	....	528	5.50
Tabeaud	86	26,400	3.57	8	20	60	345	....	553	6.44
* Necaxa	164	90,800	3.36	16	54	136	386	636	954	5.81
Cold Spring	88.5	26,200	3.34	10	20	70	320	....	512	5.79
Belle Fourche	100	33,100	3.31	15	20	65	284	656	656	6.56
Lahontan	112	40,900	3.25	12	20	80	330	580	640	5.72
Santa Maria	85	23,500	3.25	8	20	60	310	....	485	5.71
Pilarcitos	74	17,600	3.22	5	30	56	288	....	415	5.61
Morris	90	25,700	3.18	9	20	60	309	....	502	5.58
Borden Brook	64	12,700	3.10	7	24	49	280	....	337	5.27
Honey Lake	90	25,000	3.08	6	20	50	300	....	500	5.56
Goose Creek	137.5	54,900	2.90	7	16	54	304	554	741	5.39

* Hydraulic fill, wholly or in large part.

Table 58. Statistics of Successful Earth Dams

Name of dam	Location	Maximum height in feet	Freeboard, in feet	Downstream slope, 1 on	Top width, in feet	Remarks
(Principal upstream slope, 1 on 2)						
Englewood[8]	Ohio	124	16.5	2 to 4	25	Hydraulic fill. Up. slope, 1 on 2 to 4.
Owl Creek[10]	S. Dak.	122	15	2 & 5	19	6-in. layers, rolled.
Apishapa	Colo.	120	7.8	2 & 3	19	12-in. layers, rolled. Failed, see E. N. R., Sept. 13, 1924, p. 418.
Belle Fourche	S. Dak.	115	15	1.75 & 2	20	Timber core, 6-in. layers; up. slope, 1 on 2 & 3.
Standley Lake*	Cal.	113	5	2	20	Puddled core, no rolling. Up slope 1 on 2 & 3.
Ashokan Dikes	N. Y.	110	20	2 & 2.75	34	Concrete core; rolled layers, 4 and 6-in. Up. slope, 1 on 2 & 2.5.
Beaver Park	Colo.	108	.....	1.5	16	Reinforced concrete paving.
Germantown	Ohio	107	15	2 to 4	25	Hydraulic fill. Up. slope, 1 on 2 to 4.
Crane Valley	Cal.	100	.....	....	20	Hydraulic sluicing.
Pilarcitos	Cal	95	5	2	24	Puddled core. Rolled layers.
North Dike, Wachusett	Mass.	82	15	33 3 & 16 4	189	Timber core. Rolled layers.
Taylorsville	Ohio	78	19	2 & 3	25	Hydraulic fill. Up slope, 1 on 2 & 3.
Lockington	Ohio	78	16	2 to 3	25	Hydraulic fill. Up. slope, 1 on 2 to 3
Upper Crystal Springs.	Cal.	75	.....	2	....	Puddled core. Rolled layers
Wissota	Wis.	75	10	2	20	Reinforced concrete core. Up. slope, 1 on 2 & 3.5.
Huffman	Ohio	73	15	2 to 3	25	Hydraulic fill. Up. slope, 1 on 2 to 3.
South Fork	Pa.	72	.....	1.5	20	....
Dixville	N. H.	70	5	1.5	25	Reinforced concrete core.
Phelps Brook	Conn.	68	12	2.5	15	Concrete core. Up. slope, 1 on 2 & 3.
Bog Brook	N. Y.	65	8	2.5	25	Masonry core.
Piedmont	Cal.	65	...	2	...	....
Sudbury	Mass.	64	7	2 & 2.5	14	Concrete core.
Snake River	Cal.	64	.....	1.5	12	....
Glenwild	N. Y.	64	7	2.5	13	Masonry core.
Cuyamaca	Cal.	40	6.5	2.5	15	Puddled core.
Milton	Ohio	40	9.5	2	20	Rolled layers.
Bog Brook	N. Y.	25	...	2	12	Masonry core.
(Principal upstream slope 1 on 3)						
Calaveras*	Cal.	240	10	2½	25	Semihydraulic fill.
Davis Bridge	Vt.	204	.....	2½, 3, 3½	25	Hydraulic settlement from side slope. Up. slope, 1 on 3, 3.5, 4.
Necaxa, No. 2*	Mexico	190	16	2	54	Puddle core, hydraulic fill.
Terrace	Colo.	180	15	2	25	Concrete core and puddled core. Hydraulic fill.
Linville*	N. C.	160	24	2.5	20	Semihydraulic-fill. E. N. R., Aug. 12, 1920.
Goose Creek	Idaho	145	7.5	2	16	Reinforced concrete core.
Idaho Irr. Co.	Idaho	135	.....	2.5	40	....
Patillas	P. R.	135	.....	2	20	Semihydraulic fill. Up. slope, 1 on 3 & 2.
San Leandro	Cal.	125	5	2.5	28	Puddled hydraulic sluicing.
Catawba	N. C.	120	24	2.5	20	Semihydraulic fill, E.N.R., Aug. 12, 1920.
Lahontan	Nev.	120	12	2	20	Thin layers, rolled.
Dodder	Ireland	115	.....	3	22	Puddled core.
Temescal	Cal.	115	5	5	18	....
Mudduk	India	108	.....	2.5	....	....
Quemahoning[501]	Pa.	106	13	3 & 4	20	Hydraulic sluicing. Up slope, 1 on 3 & 4.
Somerset	Vt.	105	10	2.5	....	Semihydraulic fill.
Cummum	India	102	12	1	....	....
Cold Spring	Ore.	98	10	2	20	Rolled layers.
Honey Lake	Cal.	96	6	2	20	Puddled core.
Waghad	India	95	14	2	6	....
Edgelaw	Scotland	93	.....	2.5	....	....

* Slides during construction. See *E. N. R.*, May 31, 1917, p. 440.

Table 58. Statistics of Successful Earth Dams.—(*Continued*)

Name of dam	Location	Maximum height in feet	Freeboard, in feet	Downstream slope, 1 on	Top width, in feet	Remarks
(Principal upstream slope, 1 on 3).—(*Continued*)						
Sevier Bridge	Utah	92	13	4	32	Core-wall, steel and concrete.
Lewiston-Sweetwater	Idaho	85	....	2	16	Semihydraulic fill.
Yarrow	England	87	6	2	30	Puddled core.
Forrest Park	Md.	87	5	2.5	15	Puddled core.
Turdoff	Scotland	85	....	2.5	10	....
Vehar	India	84	6	2.5	24	Rock fill.
Sherburne Lake	Mont.	83	10	2	22	6-in. layers, rolled.
Cold Spring	Ore.	82	7	2	20	Hydraulic sluicing.
Roddlesworth	England	80	12	2.5	16	....
Gladhouse	England	79	10.5	2.5	12	....
Rake	England	78	....	2	....	....
Silsden	England	78	....	2	....	....
Dobbins Creek	Cal.	77	6	2	6	Hydraulic fill.
Glencourse	Scotland	77	....	....	....	....
Wayoh	England	76	....	2.5	22	....
Ekruk	India	75	....	2	20	....
Sanguijuella	N. M.	75	....	2	20	Semihydraulic fill.
Ceros (7 dams)	Spain	75 to 30	....	2	....	Rolled layers.
Leeming	Ireland	73	....	2	10	....
Lake Francis	Colo.	70	....	2	20	Hydraulic sluicing.
Zuni	N. M.	70	10	1.25	20	Hydraulic sluicing.
Upper Deer Flat	U. S.	68	7	2	20	Thin layers, rolled.
Lough Vartry	Ireland	66	....	2.5	28	....
Mukti	India	65	....	2	10	....
Stubden	Ireland	63	....	2	12	Puddled core.
Tytam Bay	Hong Kong	60	....	2	30	Masonry core.
Loganlea	Scotland	59	....	2.5	10	....
Conconully	Idaho	59	....	2	20	Hydraulic fill.
Ashti	India	58	....	2	6	....
Chollas Heights	Cal.	56	5	2	18	Steel core. Rolled layers.
Cedar Grove	N. J.	55	6	2	18	Concrete core. Rolled layers.
Merced	Cal.	50	....	2	20	Thin layers.
Rotten Park	England	50	....	2	20	Puddled core.
Vale House	England	47	....	3	18	Puddled core.
Jackson Lake	Wyo.	45	8	1.87	20	Hydraulic fill.
Lower Deer Flat	....	43	7.5	2	20	Rolled layers.
Uley Brook	England	42	....	2	25	Puddled core.
Peary	Ireland	40	....	2	12	Puddled core.
Monument Creek	Colo.	40	....	2	20	....
Swift	Mont.	40	....	1.5	20	Concrete cut-off. Rock fill against earth.
Sugar Loaf	Colo.	38	8	2	25	Rolled layers.
Cache le Poudre	Colo.	38	....	....	....	....
South Dike of Sweetwater.	Cal.	37	....	2	....	Rolled layers. Up. slope 1 on 3 to 1.5.
Little Horse Creek	S. C.	35	....	1.5	....	Masonry core.
Llanefydd	Wales	32	....	2	10	Puddle trench, 154 ft. deep.
Leroy	N. Y.	32	5	2	10	Concrete core.
(Principal upstream slope 1 on 2½)						
Little Bear Valley	Cal.	200	15	2	20	Concrete core.
Ochaco	Ore.	126	15	2	18	Hydraulic sluicing.
Tabeaud	Cal.	123	8	2.5	20	6 and 8-in. layers, rolled. Up slope, 1 on 2 & 3.
Henshaw[17]	Cal.	117	10	2.5	20	E. N. R., Aug. 30, 1923.
Dale Dike	England	102	....	2.5	12	Failed; see Ref. 14, p. 92.
Morris	Conn.	100	8.75	2	20	Concrete core. 6-in. layers. Up. slope, 1 on 2.5 and 3.
Bradfield	England	95	....	2.5	12	....
Haiwee	Cal.	91	10	2.5	20	....
Throttle	N. M.	77	8	1.5	25	Steel core.
Hinckley	N. Y.	50	8	2	20	Concrete core. Rolled layers. Up. slope, 1 on 2.5 & 3.5.
(Principal upstream slope 1 on 4)						
Terrace	Colo.	157	5	2	20	E. N. R., Apr. 7, 1921.
Druid Lake	Md.	119	5	2	60	Puddled core.
Gatun	Panama	115	30	4, 8, 16	....	Semihydraulic fill. Up. slope, 1 on 4 & 7.67.
Waialua	Hawaii	98	10	1.5	....	Timber core. Downstream part, rock fill.
Main Channel, Snake River	Idaho	86	....	1.5	20	Rock fill.

Table 58. Statistics of Successful Earth Dams.—(*Concluded*)

Name of dam	Location	Maximum height in feet	Freeboard, in feet	Downstream slope, 1 on	Top width, in feet	Remarks
(Principal upstream slope, 1 on 4).—(*Continued*)						
Middle Dam, Snake River.	Idaho	81	.....	1.5	20	Timber core. Downstream part, rock fill.
Talla..............	Scotland	78	.....	3	20	Puddled core.
South Dam, Snake River.	Idaho	66	.....	1.5	20	Timber core. Downstream part, rock fill.
Junction............	Mich.	61	12	3	12	Hydraulic fill concrete core.
Ammani Shah.......	India	61	15	2	30	....

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CHAPTER 10

WELLS*

KINDS AND COMPARISONS

Shallow vs. Deep Wells.† Wells receiving supply from deposits within 100 ft. of surface are termed shallow wells; others are termed deep wells. Shallow wells should be located in bottom lands not far from streams, may be of much larger diam. than deep wells (for advantages see next paragraph), may be dug if less than 50 ft. deep, or may be driven (see p. 232). They are obviously adapted to valleys where water plane is shallow. Deep wells must be drilled or bored as described on pp. 223 to 231. The popular notion that the water in deep wells is far below the surface is not necessarily so; a stratum at great depth may be tapped, and artesian head cause water to rise nearly to surface. Deep wells tap water-bearing strata at depths where the water is generally free from organic pollution; pollution of rock strata through fissures has been occasionally reported; the casing excludes pollution from upper strata. Water is generally hard due to dissolved gases and minerals.[1] Recession of water table will require deepening of shallow wells. Deep wells tend to depletion more quickly under pumping, due to limited intake area, the outcrop of the water-bearing stratum. Deep wells in San Luis Valley, Cal., failed due to algae growths in pipe, clogging of strata, draft beyond capacity of aquifer.[2] Iowa experience is: The deep well is costly to install, to operate, and in water treatment. Many deep wells there have been abandoned because of high mineral content‡—exceeding 1100 p.p.m.[3] Drilled wells cost less per unit of yield than dug wells, but are more costly to operate.[4] Unless there is some local factor against it, a shallow well usually is the best investment and the best water producer.[5]

Large vs. Small Wells. Storage space and an ample suction space are provided, and pumps can be handled more easily in a large well. Large wells involve a steeper slope on the cone of depression of the water-table and, therefore, cause more disturbance than several small wells. Large wells may be sunk with cheaper equipment than drilled wells. Caissons are often used (see p. 233). The yield of a well 10 ft. in diam. is but 125 per cent. more than a 6-in. well.[4] Velocity of inflow into a driven well (rarely larger than 12 in. diam.) is greater than into a large well, producing tendency to clog.[4]

Conditions for Success.§ If porous medium is homogeneous, wells need extend but few feet below water-table. Often fine sand is mixed with gravel, which soon clogs strainers. For permanent supply it is necessary to penetrate bed of coarse gravel, from which water will be freely given. In the Duryea

* See also "Construction of Wells and Bore-holes," by Dumbleton (Crosby, Lockwood, 1925).
† McGee[72] estimates from data for 7500 wells in North Central States, that average depth to water table is 17 9 ft. in Indiana, and 22 6 ft in Wisconsin.
‡ Phoenix, Ariz superseded a well supply (analyzing 1443 p.p.m. total mineral) with a surface water supply (300 p p m. total mineral).
§ See also p. 75 and p. 260.

well, Long Island, with main water-table at elevation +85, it was necessary to penetrate to elevation −145 to get coarse medium in which to stop well. Extensive drafts of water will cause a decrease in the flow of adjacent springs, and will lower the water in ponds and wells; the use of water should, therefore, be limited to an equitable quantity. System of underground supply will result in reclamation of swampy lands along brooks.[6] Never locate a well on a ridge. A hillside site is best; a well in a depression is more likely to collect drainage.

PRINCIPAL METHODS* AND TOOLS FOR SINKING†

Outline of Methods.‡ Deep wells are sunk by several methods, of which the following are the more important: (*a*) The "*Standard*" or *Oil-well* method is used for wells 4 in. to 12 in. diam., through both earth and rock, to depths of about 5000 ft. Steel pipe casing is driven down to solid rock as the hole is drilled, drilling being done by a heavy plunger lifted and dropped by machinery which breaks the material into small fragments for removal by a sand bucket. (*b*) *California or Stovepipe* method is used for wells 6 to 30 in. diam. through soil. The casing consists of short sheet-iron cylinders, forced down by large hydraulic jacks and perforated in place by a special tool. Materials within the casing are excavated by a sand bucket. (*c*) *Rotary* method is used for comparatively shallow holes; casing is rotated, and water forced down through it escapes up the outside. (*d*) In the *Jetting* method the casing is sunk by driving, while material inside is cut out by a high-pressure water jet through a small pipe, and carried to top of casing by the return water. (*e*) In the *Core Drill* method, a steel pipe casing is sunk to rock and then hole is drilled by a rotating bit attached to hollow rods. The bit cuts an annular hole, leaving a core of rock enclosed in the rod which is broken off and removed at intervals. Shallow wells are sunk by open digging (see page 233), or by drilling.

A process originating in Louisiana in 1890, is the modern hydraulic rotary,[7a] which omits casing and uses heavy drill rods to rotate a fish-tail bit. The fluid employed is clay mud, which seals and keeps intact the walls of the hole. Even coarse running gravel can be held by heavy mud, the consistence of which is controllable. This mud is forced by a slush pump through the rods to the drill. On returning upwards outside the rods, it brings the drillings with it. Since 1910, this method has been extensively used in water wells, and it is claimed to procure larger yields than other processes. Use of a rotary rock-drill bit adds to economy of the process.

Standard Method. For very deep wells a heavy *Standard outfit* is commonly used, which may be purchased of manufacturers who make a specialty of well supplies. Principal parts are a derrick tower, engine and boiler, pitman and walking beam, drilling rope of hemp or steel, bull-wheels and wheels for cable and actuating ropes, temper screw and tools. Figure 86 shows many of the tools. *String of tools* in deep drilling comprises rope socket, sinker bar,

* Well drilling firms include: Whitney Well Co., Chicago, Ill.; Ridpath & Potter, Philadelphia, Pa; Layne & Bowler, Memphis, Tenn.; Phillips & Worthington, N. Y. C.; Virginia Machinery & Well Co., Richmond, Ohio Drilling Co., Massillon, O.; American Well Works.

† Largely an abstract of U. S. Geological Survey, *Water Supply Paper* 255, 1910, by M. L. Fuller and 257, by Isaiah Bowman, 1911. See particularly 120-page article by Sands in "Handbook of the Petroleum Industry" (Wiley, 1922), p. 200.

‡ Experience of British Mediterranean Expeditionary Forces is given in "Emergency Water Supplies," by A. Beeby Thompson (London, Crosby, Lockwood & Son, 1924).

jars, auger stem, and bit. Whether or not the complete string is used depends on conditions.

Spudding is drilling without the walking beam, a method nearly always used in sinking first 75 or 100 ft. as the string of tools is too long to be operated from the walking beam in beginning work. Owing to short length of cable between tools and walking beam there is little "spring" in the rope, and hole must be spudded to a sufficient depth to allow a considerable length of cable to come between; otherwise the blow of the drill will be dead, and rope likely to break.

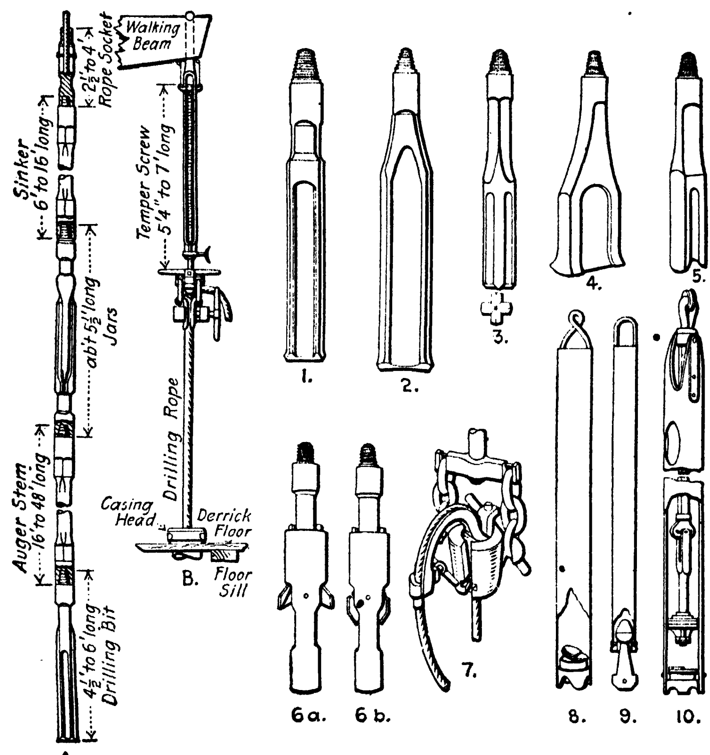


FIG. 86.—Drilling tools used with standard drilling outfit.
A. String of tools. *B.* Temper screw. 1–5. Bits. 6*a*, 6*b*. Reamers. 7. Rope clamp. 8, 9. Sand bailers. 10. Sand pump.

Casing and Drive Pipe. In deep drilling, where caving material may be encountered, it is customary to sink casing, or drive pipe, as fast as drilling proceeds.* On bottom of lower joint there is screwed or shrunk a shoe of tempered steel, which will stand heavy driving without injury, and which gives clearance for pipe and couplings.† To prevent top of pipe from being battered, a drive head is placed on it. In some places material is so loose that the tubing will follow the drill for some distance without being driven; when driving becomes necessary, drive clamps are bolted to the pin square of

* See p. 246.
† Large casings—over 18 in.—are sometimes welded; this may prove a drawback when casing must be withdrawn from an abandoned hole.

upper end of auger stem. Several machines for pulling casing are on the market.

Portable Drilling Rig. Several modified forms of the standard outfit, called portable rigs, are mounted on wheels. Horses and gas engines are commonly employed, but in some localities a traction engine supplies power for operation and hauls it when moved. Total weight of a portable rig is about 15,000 lb.; lighter outfits may weigh only 5000 lb., and heavier, 20,000 or more. Beyond 1500 ft., the stroke of most portable engines becomes so short as to make them less efficient than standard outfit.

The pole-tool or wedge-rod method of drilling differs from the standard chiefly in using wooden rods instead of cable. It is still used in the area between Illinois and Montana and north of Kansas and Colorado,* where great depth of wells and the large amount of water encountered make wood rods advantageous. In quicksand, pole-tool is almost worthless because of great time in withdrawing tools and rapid inflow of sand; it must be supplemented by rotary appliances.

The self-cleaning or hollow-rod method includes the essential features of the percussion methods, but differs in combining in one operation the breaking up and removal of material. The self-cleaning outfit is admirably adapted to sinking wells of small diam. in sand, clay, shale, soft limestone and other easily penetrated materials. The "Ohio" machine is a self-cleaning outfit fitted with a special operating device; rods are gripped by jaws and lifted by power transmitted through a crank and pitman. At upper end of stroke, levers attached to the jaws are tripped, the tools are released and fall freely, and are caught up again by the jaws on the rebound.

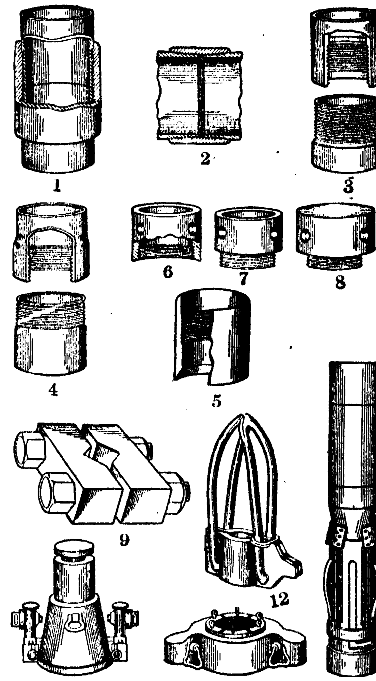


FIG. 87.—Couplings and casing attachments.

1. Sleeve coupling. 2. Tapered sleeve coupling. 3, 4. Sleeve and inserted joint couplings. 5. Shoe. 6, 7, 8. Drive heads. 9. Drive clamps. 10. Pipe ring. 11. Hydraulic jack. 12. Elevator. 13. Gas packer (used in gas wells to isolate productive layers).

California or Stovepipe Method.† For unconsolidated alluvial deposits a method is used which, on account of its origin, is called California; or is called stovepipe, from nature of its casing; also, mud-scow method, because of the instrument used for drilling and bailing. The sand bucket‡ has a flap valve and is connected to the tools by a knuckle joint that permits easy dumping, and carries a cutting shoe similar to that on bottom of a string of casing. For clay, this shoe may have a straight chisel-like bit extending as a diameter across it.

* Probably cable outfits predominate, 1925. (L. D. Conkling.)
† See also 50-page Bulletin 112, *Arizona Agri. Exper. Sta.*, by H. C. Schwalen, 1926.
‡ Same as "mud scow."

Casing is made of lap-riveted or welded cylinders of sheet iron or steel, usually No. 10 to 14 gage, 24 in. long and 6 to 16, and sometimes 30 in. in diam. Metal may be taken to the field flat and riveted there, but usually the riveting is done at shops and ends turned square and true. Two sizes are used, one of which just slips within the other, so that the joints of one may be adjusted to fall midway between joints of the other. Sections are added one at a time as sinking proceeds, each 2-ft. section adding 1 ft.; outer and inner sections are united simply by denting with a pick. Casing is watertight, but for water wells it is not necessary that it should be. Casing may be started from a properly recessed drive shoe, but it is easier to begin a well straight and keep it plumb by using a starter, 12 to 20 ft. long, of sections of stovepipe casing, often three thicknesses riveted together, or a section of heavy lap-weld casing. At lower end is an annular steel drive shoe. Casing is usually sunk by two or more hydraulic jacks buried in the ground. In small shallow wells, casing is sometimes forced down by two steel I-beams or railroad rails, arranged as powerful levers.

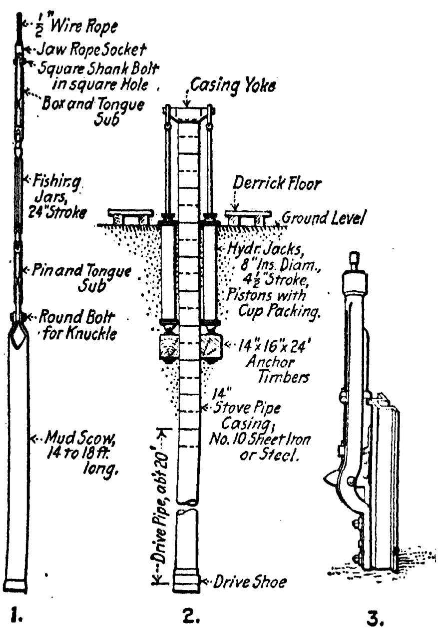


FIG. 88.—Parts of California outfit.
1. String of tools. 2. Casing and jacks. 3. Perforator.

Precautions. Water is necessary for drilling and must be supplied until struck. The sand bucket must not be filled to overflowing, or the excavated material may spill over and stone and gravel become jammed. Bowlders are worked to one side, or broken by drill substituted for the sand bucket. In quicksand or caving ground it is necessary to keep the water level about the same inside casing as on outside, also to keep casing even with, or ahead of, excavation. In very soft ground a hole larger than the casing is apt to form, and falling ground from top of such a cavity is liable to crush the casing.

Perforation of Casing. A record of material encountered is kept, and, after well has been sunk to required depth a cutting knife is lowered, and vertical slits cut in casing at selected water-bearing strata. One perforator is shown; another has a revolving cutter that punches five holes at each revolution of the wheel. Vertical slits are of a form and size that do not clog readily. For best results in fine material, a natural strainer is formed about casing by removing the finest material adjacent to pipe by pumping the well as low as possible for several days.* Sand bucket gives better knowledge of strata penetrated than "wash samples." The yield ranges from 0.05 to 5 mgd. according to conditions. Seldom or perhaps never is it necessary to re-perforate a well casing;

* See also p. 238.

the yield of an old well may often be increased by using compressed air, or by heavy pumping, loosening surrounding sand and gravel.*

Advantages of stovepipe† casing are: (1) Smoothness offers no resistance to the tools inside, and minimum to the material without. (2) If properly made, casings are not weak at joint like screw-pipe, but uniformly strong. (3) Great flexibility. (4) The short lengths of casing permit the well to be sunk by jacks of limited working range. (5) Uniform pipe allows perforations at any points that the material indicates. (6) Cheapness. (7) Hydraulic efficiency; great numbers of small holes allow for high yield with low velocity outside. (8) The absence of perforations in any part when first put down permits easy use of sand pump, and the penetration of quicksand, etc. (9) Deep wells with much screen may be drawn upon heavily with little loss of head. (10) The perforations are the best possible for the delivery of water and avoidance of clogging. (11) The large size of casing permits a well to be put down in bowlder wash where a common well could not be driven. (12) Uniform pressure by the hydraulic jacks is safe, convenient, and speedy compared with driving of casing by a weight. (13) Good progress is made in material that would be considered in many places impossible to drive in by other methods.

Rotary method‡ has been used many years for sinking shallow wells in fine-textured, unconsolidated materials, but attained prominence in deep-well drilling in 1901. A derrick like that of a standard rig is used, but machinery and tools are unlike those of a percussion outfit. Thin mud or slush plays an important part in drilling, and a slush pit, an essential accessory, is usually dug near derrick, on same side as the pumps, about 40 ft. long, 15 ft. wide, and 3 or 4 ft. deep. A ditch where sand may settle out of the mud is cut from the well circuitously to the slush pit, from which hose or pipes lead to the pumps. *Drilling* is accomplished by rotating entire string of casing with toothed cutting shoe on the lower end. Only one casing, that being revolved, is used, for as muddy water escapes upward it puddles the side of the well so that material stands alone. If clay is encountered, water used in drilling is kept as clear as possible and not drawn from the slush pit, for the clearer the water when introduced, the greater its capacity to uphold and move particles of earth, and clay is sufficiently compact to make a wall that will not cave. In penetrating sand and gravel, clay often has to be added to the slush pit, so as to make a thin mud that will plaster up these beds and prevent escape of drill water. Many clays are so compact and dry as to resist the action of water, and if the casing is fed too rapidly a core forms reducing size of opening through which the water must pass, and correspondingly increasing the pressure exerted by pumps; this may be obviated by fastening across end of shoe a bar that will cut the core. Pumps must be kept going constantly, otherwise drillings will settle and "freeze" pipes fast.

In penetrating firm material, it is sometimes necessary to employ a rotary drill bit instead of shoe; two styles are in general use—diamond-shaped and fish-tail. The diamond-shaped is usually first employed, and the fish-tail§

* See also p. 230.

† For durability, see pp. 234 and 246.

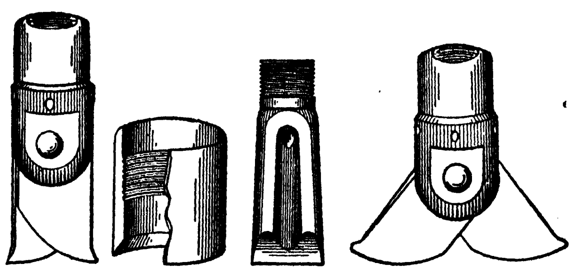
‡ See "The Rotary Method of Well Drilling," by A. G. Wolf, *Eng. Mining J.*, Aug. 2, 1919, p. 171.

§ See *E. N.*, May 13, 1915, p. 928. See also p. 228.

afterward, for enlarging. These bits are used on the smaller casing and slip down inside the larger casing. In hard rock, chilled shot or other abrasive may be used as in the shot method.

*Screening.** Many water wells sunk by the rotary method are difficult to screen because of depth at which the operation is conducted and fineness of material. One method is to puddle the wall of well at water-bearing layer, set the screen, and draw casing to top of screen. By pumping heavily for a few hours, puddled sands are partly reopened, but the method has the defect of leaving water-bearing layers more or less clogged by fine material. A more difficult, but better and more common, way is to sink the screen below the casing by forcing a hole down by a jet of water, the wash pipe being run ahead. A packer or lead seal is then inserted at the point where top of screen joins well casing to prevent materials from rising over top of screen and filling it.

Jetting Method. In the jetting method, material is both loosened and carried to the surface by water under pressure. The principal parts of the outfit are force pump and water swivel, drill pipe, nozzle or drill bit, casing



1 2 3 4

FIG. 89.—Jetting outfit.

1. Paddy (or expansion) drill closed. 2. Drive shoe. 3. Common jetting drill. 4. Paddy drill in widest position.

and drive weight. Water is led into the well through a pipe of relatively small diam. and forced downward through the drill bit against the bottom of the hole. The stream loosens material and the finer portion is carried upward and out of the hole. The drill pipe is turned slowly to insure a straight hole. Casing is usually sunk as fast as drilling proceeds. In softer materials, by using a paddy or expansion drill (which opens when it strikes bottom; Fig. 89, 1 shows closed position, Fig. 89, 4, open), hole may be made somewhat larger than the casing, which may be lowered a considerable distance by its own weight. Ordinarily, however, a drive weight is necessary to force it down. As a rule, one size of casing may be employed for entire depth. It is usually difficult to drive a single casing beyond 500 or 600 ft., and if well is much deeper, a smaller size must be used. In fine-textured, clayey, or loamy material, hole may be jetted down to full depth and casing inserted afterward.

The jetting method is much employed for putting down wells in the Atlantic Coastal Plain and in some of the valleys of the arid West deeply filled with alluvium. In Coachella Valley, southeastern California, the method has been successfully used for flowing artesian wells. Wells were usually 400 to 500 ft. deep, 4 in. in diam.; not uncommonly a well may be sunk, cased, and cleaned in two days.

* See also *Screens* p. 237.

Core drills are little used for sinking wells, though tried from time to time; diamond drills have been employed to some extent in South Africa for deep water wells. The core drill principle is, however, occasionally employed in connection with more common well-drilling outfits. All rotary core drills are portable, can be taken apart for transportation on pack animals, and used where more cumbersome outfits are debarred. Nearly any power can be used—electricity, compressed air, steam, gasoline, horse, or hand.

Diamond Drills. Diamonds must be selected with special attention to uniformity of size and weight, as irregularity will disturb the balance of stones and necessitate frequent resetting. They usually require resetting

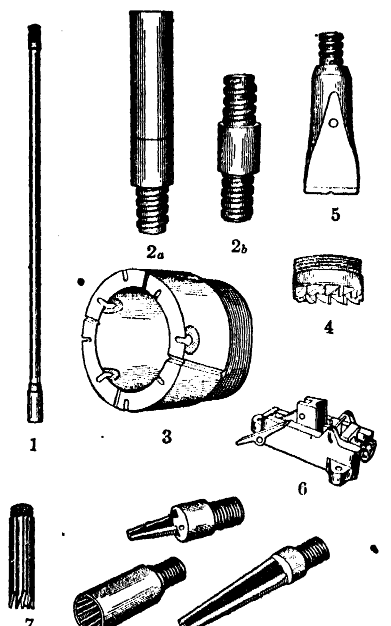


FIG. 90.—Parts of core-drilling outfit.
1. Davis calyx drill rod with coupling attached. 2*a*, 2*b*. End of drill rod and rod coupling. 3. Bit to be set with diamonds. 4. Toothed cutter bit. 5. Chopping bit. 6. Safety clamp (prevents loss of drill rods when hoisting out). 7. Toothed collar. 8. Coupling recovery taps.

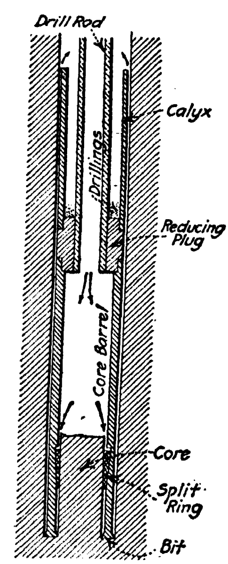


FIG. 91.—Calyx core barrel.

after 8 to 12 hr. work. The size of diamonds ranges from about 1 to 4 carats, according to size of bit. Stones of approximately cubical shape are best, as they are stronger and furnish better cutting faces. In starting, the first requirement is to get down to rock. If the soil is thin, a pit is dug, drive pipe inserted, and tight joint made by chiseling a seat in the rock, driving the pipe down, and calking it firmly. If soil is more than 10 or 12 ft., it is cheaper to drive the pipe to rock. Where drive pipe cannot be seated with sufficient firmness to keep out surface water, a hole is drilled inside with a chopping bit, and a string of casing put down to a depth sufficient to exclude water.

Calyx Drill. The hoisting and driving machinery of the calyx drill is similar to that of the diamond drill, and feed water is supplied through a

swivel and hollow drill rods. The bit is of hardened tool steel and consists of a toothed collar, somewhat like the cutting shoe of the hydraulic rotary outfit, but having a longer barrel and teeth, the teeth being so set as to provide clearance to the core and to the bit and rods. Above the core barrel a cylindrical chamber, or calyx, open at the top, encircles the drill rods. In it coarser rock fragments torn off by the bit are caught as dropped by the upward water current when its velocity decreases. They are removed when the rods are hoisted, and furnish a second record, in inverted order, of material penetrated, of especial value in material too soft to yield a core.

Chilled Shot Method.* Experiments in drilling by loose abrasives poured down the drill hole led to adoption of chilled steel shot, such as is used in sawing stone. Other parts of the shot outfit are similar to the diamond drill, but cutting is accomplished by revolving an iron or steel tube on the shot. A slot a few inches long and half-inch wide, cut into the lower end of the tube or bit, allows shot to reach the cutting surface more readily and be more evenly distributed. Distribution is also aided by slightly beveling the edges of tube so that shot may get under it. Under weight of the drill rods the shot bites into the rock and chips or wears off small pieces, which are brought to the surface by the water current.

Precautions in sinking important wells: (1) Accurate log should be kept, so that depth and character of water-bearing formations may be known. (2) Every water-bearing layer should be carefully examined as to thickness and quality of water. (3) Head of water of each water-bearing layer and its relation to other water encountered should be accurately determined, so that, if necessary, contamination may be prevented by using packers and separate pipes for each water horizon. (4) Casing should be intact when well is completed, and should be kept so in order that it may fulfill its duty in shutting out undesirable water. Its condition should be determined from time to time by suitable experiments. (5) Possible effects of defective casing should always be considered in interpreting a change in head or quality of water. (6) To exclude surface water tie a leather "seed bag" filled with flaxseed firmly around the pump tubing and let down to proper depth. In a few hours the seeds swell and fill space between tubing and wall of well.

Increasing Supply. Explosives are used to some extent for increasing supply and providing a reservoir;† 190 quarts of nitroglycerine exploded in an 8-in. well at Kennett Square, Pa., increased the yield from 3.3 to over 24 g.p.m.[9] If water is drawn from rock, fissuring the rock will increase the area from which delivery is made. In a limestone region, where the underground water, like the surface water, runs in more or less definite channels, instead of percolating slowly as a broad or thick sheet, torpedoing will almost surely increase the number of contributory veins. The steam jet is sometimes used in unconsolidated deposits. Steam is forced down a small pipe inside of a larger one, and coming into contact with water at the bottom turns it quickly into steam, the resulting explosion loosening the material or making a pocket about the bottom of the pipe. Where the materials are dense and

* For details of British practice, see Dixon, *Proc. Inst. of Water Engrs.*, 1923 (Abstracted in *E. C.*, February, 1924, p. 347).

† See "Shooting Wells to Increase the Flow," by S. R. Russell, DuPont representative, *Munic. County Eng.*, February, 1922, p. 70.

clayey, action of the jet may considerably increase the influx of water; in more porous deposits, it has less effect. Deepening a well beyond a certain limit will neither increase the supply nor add to the hydraulic head, but it may increase the mineral content to the point of abandonment. Back-blowing with compressed-air often increases supply.

MINOR METHODS OF WELL SINKING*

Introduction. Shallow wells are generally sunk by these methods. Dug wells require least equipment but most careful supporting of sides. Dug wells seldom exceed 50 ft. in depth. If tightly lined, inflow is limited to the bottom. These are termed "percolation" wells in India.

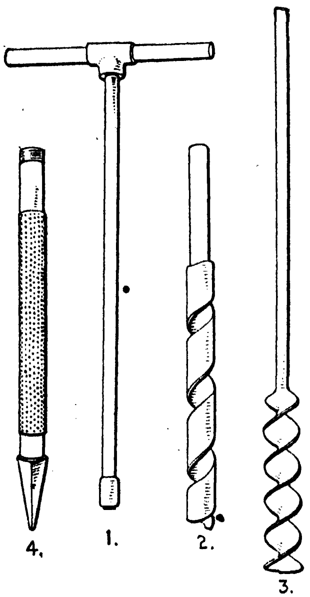


FIG. 92.

1. Small earth auger. 2, 3. Earth auger bits. 4. Drive point and screen.

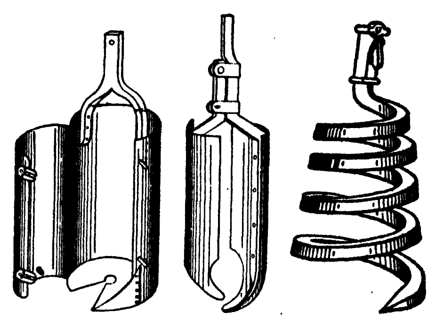


FIG. 93.—Tools for auger borings.

1, 2. Ordinary clay auger, (3 to 36 in. diam.). 3. Special auger for penetrating small bowlders or soft rock.

Boring with Auger. In alluvial and other unconsolidated deposits, wells 2 or 3 in. diam. are in some parts of the country bored to ground-water level with a hand auger made by welding a carpenter's auger to a rod or pipe. Auger works more efficiently if the centering point is cut off and lips are shaped as in Fig. 92 (3). As the auger is heavy when loaded, a windlass or small derrick may be used in lifting. In boring through dry sand or other loose deposits a little water should be poured into the hole to cause the material to cling to the auger. Casing is not usually required until the auger reaches saturated sands; it may be driven by a wooden maul or ram. Mud and water-saturated sand may be bailed out with a sand bucket. Hard layers may be penetrated by a drill similar to that used with percussion outfits. Where the ground holds together well an Arkansas clay auger is employed for sinking wells to depth of several hundred feet, especially in lower Mississippi Valley. This auger is 15 ft. long, and consists of a cast-steel barrel 4 ft. long, which resembles a 3-in. pipe sawed vertically in half. This is fastened by a flat piece of iron to a second auger barrel, $1\frac{1}{2}$ to 2 ft. long; above this is a second piece of flat iron, square at the top, and cut with threads for fastening to wooden poles. At the bottom of the auger barrel, on the right side, is riveted

* See second foot-note, p. 223.

a steel cutting edge, commonly called the cutting bit, projecting inward 1¼ in. On opposite side, and slightly above, is the "auger lip," which helps to hold the dirt in the auger barrel, when the tools are lifted. The auger is fastened to a 10-ft. auger pole, and this to the regulation 26-ft. pole. The tools are turned with a clamp, and when the bit begins to choke, the tools are lifted and dropped by means of a windlass. This operation jumps the dirt in the bit, and so frees the lower end; it is termed, "making a slip." If the clay is very dry a little water is added, and with very sticky Cretaceous clay this process can be continued until the whole length of the auger is filled. This 15 ft. of mud represents about 10 ft. in depth. Usually the auger is filled for only about 10 ft., representing 7 ft. depth, before lifting the tools. When rock is encountered, a bar drill is used, sometimes attached directly to the wooden poles and sometimes to iron poles. When sandy layers are encountered, which will not hold in the auger, a sand pump is used, or enough clay is dumped into the hole to make the sand stick together. Where wells of larger diam. are desired, the hole is enlarged with a reamer.[10]

Punching. In a few localities where material is very clayey, wells are sunk by a punch. This method, used in Arkansas and Louisiana, employs a cylinder of steel or iron 1 to 2 ft. long, split along one side, and slightly spread. The lower portion, very slightly expanded, sharpened, and tempered into a cutting edge, is attached to a rope or wooden poles and lifted and dropped in the hole by means of a rope given a few turns around a windlass or drum. Material is forced up into the bit, slightly springs it, and so is held. When working in very dry clay, water is sometimes added. Thin sand layers are passed by throwing clay into the well and mixing it with the sand until the bit will take it up.[10]

Driving Pipe. Where the water level is within the suction limit of about 25 ft. below the surface, and water-bearing beds are unconsolidated, small supplies may be cheaply obtained by driving a strainer and drive point fastened to a piece of pipe. When desirable water is reached, the well is pumped rapidly to free the strainer from particles of sand and clay. The coarser material adjacent to the strainer is thus washed free and forms a natural filter. British Army officers call it the Abyssinian well, as it was first used extensively during the Abyssinian campaign of 1895. Clay and sand often cling to the screen and render it worthless. To obviate this, pipe may be driven to the required depth with only a drive point and then pulled up a distance equal to length of screen. The drive point, which fits loosely on the pipe, remains behind, and the screen is lowered upon it.* This style of screen can be removed and cleaned if it becomes clogged, but can be successfully used only when depth to water-bearing layer is known, since a test for water cannot be made until the pipe is raised. A modification consists of a screen attached to a drive point and so arranged that the drive pipe slips down over the screen and rests on the point while being driven. In testing for water, the pipe is withdrawn a foot or two, and a pitcher pump is screwed to its upper end. If water is not procured, the pipe is driven again. The only drawback to this is that pebbles may lodge against the screen when the pipe is withdrawn, and when it is driven down again, may tear the screen. If the water contains much iron, a thick crust may form on the pipe and screen and reduce the inflow of water or even shut it

* See Fig. 92, p. 231.

off entirely. Incrustations may be loosened and broken up into fine particles that may be pumped out by pulling the pipe a few inches, by driving it down a short distance, or even by rapping the pipe sharply. While cleaning a new well, pumps should not be stopped until sand is no longer discharged, otherwise sand will settle about the valves of the pump and make it impossible to start again without drawing the pump rods and valves, cleaning them and resetting. If the well is sunk into unconsolidated materials, a screen* must be used which will permit water, but not sand in serious quantities, to enter.

Objections to Driven Wells. Driven wells, as a continuous source of supply, are open to the following objections: (1) The extreme difficulty of determining the tributary area prevents a satisfactory estimate of yield and for the same reason it is difficult to remedy pollution, or trace its sources. (2) Doubt as to the permanency of the supply; and possible deterioration. (3) Legal complications arising from the effect on adjacent catchment areas. (Municipalities are liable for damages where the water-table has been lowered by diverting water for public use.) (4) Infiltration of sea water, unless the wells are most carefully managed, is imminent near the ocean.

Large Dug Wells. A well at Greenfield, Mass.,[11] is more or less typical. This well is 40 ft. diam., 30 ft. deep, 25 ft. of which is below ground-water level. Excavation was made to water level, then forms were built and a reinforced concrete wall, 24 in. thick at top and 30 in. at bottom (battered), placed to height of 10 ft., 5 ft. projecting above ground. Wall was sunk by digging beneath it. As the wall settled, concrete was added to its top. The bottom of the wall was beveled inside to serve as a cutting edge. As the concrete was placed, many 2½-in. tile pipes were placed in it to permit free passage of water through the wall. A 6-in. centrifugal pump, belt-connected to an electric motor, was mounted on brackets bolted to the inside of the wall, during construction. Since the pump sank with the wall, it was not necessary to lower the suction pipe. The discharge pipe was so constructed that it would turn on an elbow outside the wall as the well sank, without straining the connections. There is a reinforced concrete dome over the well, with a manhole over the suction pipe. The dome is covered with 2 ft. of earth. The well is situated in a bed of coarse sand and gravel close to the Green River, and was pumped during 1914 at rates of 1.2 to 2 mgd. It is used only in emergency, as an auxiliary supply (1925).

A well near Banning, Cal., was sunk 100 ft. below water-table by means of a shield, similar to river-tunneling type.[7b] Cost data on sinking a caisson, 25 ft. in. diam., 45 ft. deep, at Chillicothe, Ohio, are given in *E. N. R.*, Sept. 6, 1917, p. 460. At Wampaca, Wis., the caisson was 30 ft. inside diam., and 37.5 ft. long.[62]

WELL CURBS, CASINGS, AND COVERS†

Dry rubble curb and casing‡ utilizes all seeps. The material costs little; little money outlay for labor. The well is never safe near sources of contamination. Affords no filtration and allows dirt and soil to enter. Permits entrance of mice and other small animals at the top. Do not use stones par-

* See p. 237.
† See also p. 630.
‡ Such linings have been made tight by applying gunite; see *E. C.*, Oct. 10, 1917, p. 291.

tially or wholly covered with moss or lichen; persistent impairment of the quality of the water has resulted from such use.

Dry brick curb and casing utilizes all seeps. Filters out most sediment. Does not allow small animals to enter. Involves little money outlay for labor. Polluting matter enters readily, and the well is never safe near sources of contamination.

Curb and Casing of Masonry in Cement. Water from bottom only is utilized. Entrance of sediment and animals is prevented. Wall does not impart taste to the water. The well is safe from pollution (except that entering at bottom), as long as walls are not cracked. Well is unsafe if so shallow that polluting matter can reach its bottom. Costs more than uncemented wells; may require skilled labor. A common practice is to use cement in upper part but not in lower, where water of satisfactory quality is admitted.

Wooden curb and casing is cheap in many localities. Can be used in wells of small diameter. It imparts, when sound, no taste to the water. It swells tight in wet ground, water either entering at bottom, or (after sudden rises), through shrunken portion at the top. Pollution enters readily, and animals gnaw through. The wood rots, giving taste to water and favoring development of bacteria.

Glazed and Cement Tiles. (1) With uncemented joints, utilizes all seeps; imparts no taste to water; requires no skilled labor; polluting matter enters readily and well is never safe if near source of contamination; soil may wash in through joints. (2) With cemented joints, well is safe from pollution (except that entering at bottom) as long as joints are tight; does not require expensive labor; can be used only in soft materials containing considerable water. Used on 24-in. well at Medway, Mass.[39]

Metal casings are adapted both to rock and to unconsolidated materials. Safe from pollution except that entering at bottom. The cost in large, deep wells is considerable. Use is practically limited to wells under 14 in. diam. Casings have been subject to deterioration by corrosion and incrustation; this led to use of ingot iron and cast iron. At Tunbridge Wells, England, cast-iron pipe was substituted for steel; diam., $32\frac{1}{4}$ in., thickness, $1\frac{1}{4}$ in.; length 12 ft. Slotted cast-iron tubes were used for strainer. Tubes were coated by Angus Smith process.[36] A disadvantage is the greater weight to handle; a Chicago firm facilitates lowering the casing by refilling the drilled hole with sand on which the casing is erected. As the sand is pumped out the casing gradually settles.[8] The diameter of the finished well should be specified, so that the driller may choose his starting diam., to allow for reducing size of casing as conditions require.*

Concrete Wells, Holland, Mich.[40] Well, depth 72 ft., of concrete pipe† from surface to bottom; inner diam. 25 in., outer 32 in.; bottom part of strainer sections about $12\frac{1}{2}$ in. long with vertical keystone-shaped grooves cast in outer surface. Water flows into grooves, down to bottom of section, and into well. Sections are held apart approximately $\frac{3}{32}$ in. by bosses cast on their ends. Temporary steel casing was first driven entire depth and material removed by an orange-peel bucket. A concrete plug was then lowered to the bottom by four steel cables. Strainer sections were then lowered, threaded over the steel

* See p 224.
† Patented by Kelly Well Co., Grand Island, Neb.[40]

cables, which passed through vertical holes cast at the quarter points of each section. Plain pipe sections followed similarly. Space of about 5 in. between concrete and steel casings was filled with gravel to top of strainer sections and with clay for remainder. Steel casing was withdrawn during filling. A 24-hr. test was made at continuous rate of 2000 g.p.m., maximum capacity of pumps available; during first 8 hr., water lowered continuously but remained constant thereafter, 32 ft. below original level. Cost, including test, but excluding pumping equipment, was $4728.

Combination dug and drilled well is particularly dangerous, because of the fancied security. A drilled well is sunk in an old, dug well, the casing commonly beginning at the bottom of the old well. The casing should be carried to surface of the outside ground, or at least above highest water level, or the dug well should be converted into a watertight cistern.

Covers. Open wells should be protected by a watertight iron or cement cover standing somewhat above the surrounding ground, and tightly joined to the curb; sloping the earth away from the well serves to run rain-water or pump drippings away, so that little will penetrate, even if the curb becomes cracked by frost. Except that it keeps the larger animals out, ordinary plank covering affords but little improvement over the open well. Crevices almost invariably exist through which small animals find access, and dirt washed through the cracks is of the most dangerous kind, of filth from domestic fowls, and from the shoes of farm hands (see Fig. 250, p. 630).

OVERCOMING DIFFICULTIES IN WELL SINKING*†

Locating Lost Tools. First step is to learn shape of upper end and position in well, by lowering over the tool a sheet-iron vessel containing soap or other soft material, in which an impression is easily made, or if above water, or in a dry hole, by reflecting light into the well from a mirror. A photographic, stereoscopic device, invented by Loran, a Baku engineer, is lowered to a point near the lost tool, light being furnished by an electric current carried by wires in the camera. Good photographs made in this way are given in A. Beeby Thompson's, "The Oil Fields of Russia," (Van Nostrand, 1904), showing exact shapes and positions of fallen tools.

Bowlders, if especially hard, may be blown to pieces by dynamite or rock powder tamped with a bushel or two of dry sand or clay, so that the casing will pass down between the parts; or broken into pieces so small that they can be further reduced by the drill and removed by the bailer. Casing should be drawn 3 or 4 ft. above the charge.

Running Muds and Clays. Mud produced from some shales hardens quickly when exposed to air. The hole must be cased and drilling must be pushed so rapidly that the mud will not have time to solidify. The drill must be freed from this mud and withdrawn by slowly working it up and down so as to gain on the upstrokes, and the mud may be removed by small buckets or augers. If this method fails, $1\frac{1}{2}$- or 2-in. pipes may be lowered and the hardened mud and sand flushed out by a powerful water jet.

With only a small quantity of water, clay will "crawl" and relieve pressure by squeezing through very small openings in threads or sheets. Slow

*See second footnote p. 223.
†See methods used at Oglesby, Ill., in *E. N.*, Sept. 2, 1915, p. 450.

but forcible movement of plastic clay into a drill hole may fill the hole during a single night, when drilling is suspended; next morning the drill will strike this soft plug and ram it down until compression of the air below prevents its further movement. Drill may pound on this cushion for days or weeks without progress, while clay slowly accumulates in the hole. This difficulty may be overcome by casing off the clay before it forms a plug, or by jetting through the plug. Plastic clays are encountered in South Dakota and Atlantic Coastal Plain, but most glacial clays are so sandy that they yield readily to the drill even if the well becomes clogged.

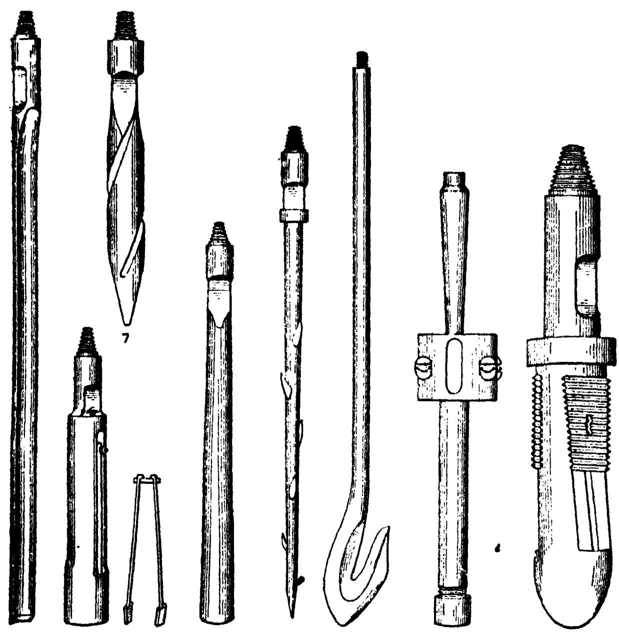


FIG. 94.—Fishing tools used with standard drilling outfit.

1. Spudding spear (prods loose a jammed drill rod). 2. Slip socket. (Teeth on slip project upward to grasp tools.) 3. Horn socket (slit on opposite sides to grasp tools near surface when jammed over them). 4. Rope spear. 5. Rope knife. 6. Casing cutter. 7. Pipe swedge (straightens casing). 8. Slip socket for inside of pipe.

Quicksand. In the coastal plain from Cape Cod westward and southward along the border of the continent the most serious difficulty is caused by quicksand, as a rule interstratified with coarser sand and clay. Quicksand comes into the hole and must be bailed out in large quantities before the casing can be driven farther and drilling continued. Under ordinary conditions quicksand will not yield its contained water, and, therefore, if it has a tendency to rise in the pipe, the difficulty can seldom be obviated by pumping alone. Pockets or lenses of clay or coarse sand in quicksand may cause the driller to think he has passed through the quicksand. Coarse sand, such as "bar" sand, will not rise if the velocity of the water through it is less than about 2½ ft. per sec. Drive pipe shuts off water and quicksand above a pocket of coarse sand or clay, but as soon as the drill penetrates the pocket quicksand flows in and may rise to top of deposit. If the bed is 20 ft. or more thick, the

pipe cannot be driven through it on account of the resistance of the compact sand; and if the water in it is under great head, so as to force the sand up to or above the point at which the bed was struck, further progress may be almost impossible. In some wells, quicksand has risen 100 ft. above the depth at which it was struck. If the hole is not kept full of water, pressure exerted by quicksand on well casing may be great. Experiments have shown that quicksand partially saturated with water exerts a lateral pressure equal to half its vertical pressure. Beyond point of saturation pressure is hydrostatic. If quicksand is only a few feet thick it may be penetrated by bailing and then driving the casing; this pipe is driven as far as possible into the bed without bailing, and quicksand may occasionally be passed through at one drive. A thin bed of quicksand near the surface may be shut off by sheet piling. Stones, clay, and asphalt have been dropped into the hole to restrain quicksand, with some success. Some drillers maintain that quicksand can always be penetrated by keeping hole full of water. If working in sand of fine texture, draw the drill at night, as otherwise sand may creep up around the drill and "set" almost as hard as rock.

Deflection of Drill-hole. In beginning a well, care must be exercised to make the hole plumb; otherwise drilling to depth is difficult, if not impossible, because of friction of the tools caused by increasing deflection. To keep the hole straight the driller may lengthen tools to 60 or 80 ft., chiefly by using a long auger stem. If the drill hole gets badly deflected, the only pump adaptable is the air lift.

EXCLUDING SAND*

Screens or strainers are perforated cylinders designed to exclude sand from working barrels of deep-well pumps. They (1) prevent cave-ins of the porous medium; (2) admit water to the well. They may be of metal or concrete.† A screen in use in the Mississippi Valley from St. Louis to the Gulf is made by wrapping No. 14 wire, 10 to 14 wires to the inch, around perforated wood or iron piping; the closeness of the wrappings depends on fineness of the material to be screened out. Wooden piping as a base for a screen is often preferred on account of rusting of iron pipe. In some wells these screens are 100 ft. long. The weight of the wire screen and the sand pressure against it when set prevent the buoyancy of the wood from lifting the pipe.

Strainers may have either: (1) Fine openings to exclude the finest sand; or (2) openings somewhat larger than the finer particles in the water-bearing material, and smaller than the coarser. Pumping soon exhausts the finer material next to the strainer, and the coarser particles form what is really a greatly enlarged strainer outside the metal one. If no coarse particles exist, ram some down outside of casing (see p. 238). Such a well cannot clog. Any strainer depending on wire gauze or thin, perforated iron to exclude fine sand is undesirable, for friction is high in small openings in sand and screen; and corrosion will close up the small holes in the metal.[31]

Several patented brass strainers are used. The Layne‡ strainer differs

* See also "Lost Supplies," p. 243.
† Patented by Kelly Well Co., Grand Island, Neb.[60]
‡ Layne & Bowler Co., Memphis, Tenn.

from shop-made ones in the shape of wire. The Cook* well screen consists of a single piece of seamless brass tubing, perforated by horizontal slits, in different widths for different sizes of material, cut with a beveled edge on the inside. The screen may be set in the bottom of the well, the pipe drawn until it is almost flush with the top of the screen, and a lead seal inserted at the joint, or the screen and pipe may be united by a screw coupling. Telescoping strainers are used when withdrawal of the casing is impracticable. Johnson† well screens are made from brass containing over 70 per cent. copper to increase resistance to corrosion. The brass pipe is slotted continuously by a spiral cut, and is reinforced by longitudinal rods soldered on the inside. The makers cite tests at the University of Minnesota to show that, size for size, the Johnson well screen has from 30 to 60 per cent. greater capacity than any other brass screen.

Packing with Gravel or Coarse Sand. Frequently, where the material is very fine, it packs around the well so as to hinder entrance of water. It is common practice to drop pebbles into the well, and, with the aid of a drill, force them into the surrounding clay, etc., until a pocket is produced through which water flows freely. In quicksands, the space between the outer casing and the pump tube and strainer is often filled with coarse sand through the heavy suction induced by pumping. Many materials yielding water consist of mixtures of sand or gravel and clay. By heavily pumping a new well, it is often possible to remove fine clayey material, leaving sand grains and pebbles in a pocket about the well. A similar method is employed in certain stiff clays in which small open pockets at the bottom of the well are apparently produced by heavy pumping. It is sometimes necessary to introduce gravel through auxiliary holes drilled near the well and terminating in the top of the sand layer. Pumping at an excessive rate removes the sand and allows the gravel to settle next to the metal, forming an exterior gravel screen. Gravel should be clean and practically round, varying from ¼ in. up to 2 in.[32] A cylindrical shutter screen, rather than a fine-mesh gauze screen, gives better results with gravel packing.[61]

HYDRAULICS AND YIELDS OF WELLS

Size.[12] Inside diam. of well pipe is determined to smaller degree by quantity of water to be raised than by consideration that in sandy soil velocity at which water enters pipe must be below a certain value to avoid clogging. If H is height of perforated pipe in zone of underground water, V permissible velocity with which water may enter pipe, D inside diam. of pipe, $Q = DHV\pi \div C_p$, and $D = C_pQ \div HV\pi$, where C_p is coefficient of permeability. V is to be determined by pumping experiments; if $V = 0.0033$ ft. per sec. and $C_p = 4.0$, $D = Q \div 0.00259H$. If $H = 6.5$ ft. and $Q = 0.014$ cu. ft. per sec., $D = 0.83$ ft. If a suction pipe with external diam. D_1 is inserted in well, $D = 1.2D_1$, and $D_1 = 0.833D$.

Arrangement and Spacing.[12] Several wells may be arranged in a straight row perpendicular to flow of underground water, on right and left of line of most rapid drop of water, wells being so spaced that full utilization of under-

* A. D. Cook, Lawrenceburg, Ind.
† Edward E. Johnson, Inc., St. Paul.

ground stream is assured; or in two or three rows with greater distances between wells; or in a circle.

In a single row the distance between wells equals $\frac{3}{4}D_c$, where D_c is diam. of circular area from which water for each well is collected. Length of area from which water is collected is, therefore, $L = (n - 1) \times 0.75D_c + D_c$, or approximately $0.75D_cn$, where n is number of wells in one row. The width of this collecting area is D_c; hence area $A = 0.75nD_c^2$; for $n = 10$ and $D_c =$ 164 ft., $A = 202,000$ sq. ft. For double-row arrangement (wells staggered) distance between wells longitudinally is $1.5D_c$, and in cross direction $1.25D_c$, and in this case length of area from which underground water is collected is $L = 1.5nD_c$. Width of area is here $2.25D_c$, and therefore area of whole collecting district is $A = 3.375nD_c^2$, and if $n = 5$ and $D_c = 164$ ft., $A =$ 454,000 sq. ft., that is, more than twice area with single-row arrangement with same number of wells, so that capacity of wells is increased. With three-row arrangement, distance between wells in longitudinal direction is $2.25D_c$, and in cross direction $1.25D_c$, so that length of collecting area is

$$L = (2.25n + 1)D_c$$

and width $W = (2 \times 1.25 + 1)D_c$; therefore area

$$A = 3.5(2.25n + 1)D_c^2,$$

Hence for $n = 3$ and $D_c = 164$ ft., $A = 3.5\ (2.25 \times 3 + 1) \times 26{,}896 =$ 730,000 sq. ft.

In a circular arrangement of wells, the length of collecting area perpendicular to underground stream is same as with row arrangement, namely, $0.75D_cn$, where n is number of wells on one side of center line. With respect to direction of underground stream such a ring may be considered equivalent to two rows, upstream half of circle forming one row, downstream the other. With respect to these rows wells are arranged at a certain distance apart. Collecting area is a ring the mean diam. of which, D_m, is the average diametral distance between centers of wells, opposite each other. Outer diam. is $D_m + D_c$, inner diam. $D_m - D_c$; hence $A = D_mD_c\pi$. Circular area enclosed within this ring is not subject to collecting effect of wells. For $D_m + D_c =$ $0.75D_cn$, $D_m = 0.75D_cn - D_c$; if $n = 10$ and $D_c = 164$ ft., $D_m = 1066$ ft.

It is often poor economy to place wells nearer than 50 ft. apart, and at times even 100 ft. may be the suitable distance.[13] This refers to small driven wells.

Table 59. Interference of Wells

Theoretical Mutual Interference of a Group of 6-in. Wells, with Radius of Circle of Influence = 600 ft., Pumped Down 10 ft. (Slichter)

Spacing, ft.	Interference, per cent.		
	Two wells	Three wells	Large number in row
5	38	55	————
10	35	51	————
100	20	31	66
200	16	22	45
400	11	12	24
600	————	————	14
1000	6	8	6

Hydraulic requirements to be satisfied by spacing are: Diameters and lengths of strainers must be such that the loss of head due to friction in them will be immaterial compared to loss in water-bearing stratum. The wider the spacing, the greater is the yield to be handled, and the greater the loss of head in pipe and strainer. Forty-two wells were sunk for Memphis Waterworks, ten being 6 in. in diam., and others 8 in. Depth varied from 260 to 480 ft., of which 80 to 130 ft. were in water-bearing strata. A 75-ft. spacing was found inadequate, and 250-ft. spacing was used in later wells.[15] Experience at Savannah teaches that there is less interference when wells are on an axis perpendicular to the line of flow, than when placed promiscuously.[16]

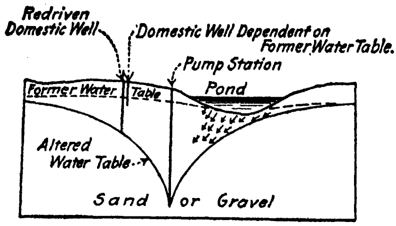


FIG. 95.—Cone of depression produced by a pump station and its effect on a near-by pond and well.

Yield depends on transmitting capacity of the soil, on extent of supply, on size of hole and rate of pumping.[20] Boring is usually continued some distance below where water is tapped, in order to strengthen the supply, and the pump cylinders are placed some distance below the water level to allow for lowering by pumping. Yield of large, shallow well may be increased by a system of infiltration galleries leading to it.*

Specific capacity is a numerical expression of the readiness with which a well furnishes water to a pump, and depends on the coarseness of the medium and the resistance offered by the strainer. The quantity can be computed by dividing the yield of the well by the amount that water is lowered in the well.

Miller-Brownlie formula† for cone of depression, as adopted by Horton:[19]

$$t = 2.30 \frac{BQ}{2A\sqrt{AQ}} \log_{10} \frac{\sqrt{Q} + h\sqrt{A}}{\sqrt{Q} - h\sqrt{A}} - \frac{Bh}{A}. \tag{1}$$

$$A = \frac{I\pi}{12m^2};\ B = \frac{P\pi}{m^2};$$

$$Q = \frac{\pi I}{12}\left(\frac{h_g - h_t}{m}\right)^2. \tag{2}$$

where t = elapsed time, years, since depletion began.

Q = rate of draft, cu. ft. per yr.

h = the cone depression of the water-table in ft. = mr = average slope (m) × radius (r), of cone of influence at end of "t" years.

I = infiltration depth, in. on surface per year.

P = available porosity of the medium, in per cent.

h_g = total depth of ground water, ft.

h_t = "trumpet head," or depressed apex of cone, measured below h, in ft. This is the only term varying with well diameter.

TESTING AND DEPTH MEASUREMENTS

Testing is measuring the rate of flow. It is considered a part of the driller's duty, and is so stipulated in well contracts (see p. 232). Wells in operation

* See also p. 230.

† See also N. Werenskiold in *E. N.*, Aug. 10, 1916, p. 257.

are often tested to detect leakage through casings. Piezometers and current meters were employed on the artesian wells at Honolulu.[24] Usual method of testing is by pumping rapidly for several hours, and metering the discharge or counting pump strokes. Wells sunk by percussion methods may be tested by bailing rapidly. Testing involves a knowledge and interpretation of the geologic structure. Sometimes a driller drills a well 400 or 500 ft. deep into "dry" rock; surface water drains into it, and the "test" consists in pumping accumulated surface water from a deep drill hole.

Defective Flow.[25] Suppose two porous beds, *A* and *B* (Fig. 96), separated by an impervious layer, are to be tested, and testing of *A* has been neglected.

Suppose seed bag or rubber packing placed above the upper one. If both bear a water level equally high the test will be fairly made and the result will indicate their combined capacity; or, if both heads are at least as high as the

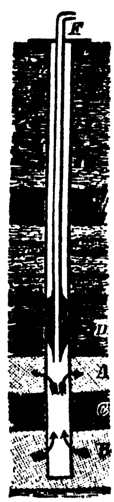


FIG. 96.—Negative test.

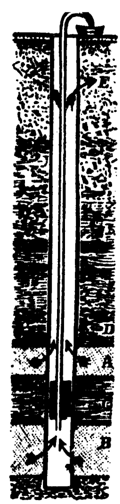


FIG. 97.—Partial and misleading test.

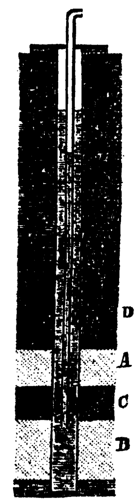


FIG. 98.—Inverted test.

surface at the well, the test may be accepted. But suppose bed *A* has been cut into by erosion or been reached by crevices, or is otherwise defective, while *B* remains intact and bears an elevated fountain head. Under these conditions water may flow from *B* through the bore into *A* and escape; in this case, result may be either simply negative (Fig. 96) or positively false and misleading (Fig. 97). If lateral leakage through *A* effectually disposed of flow from *B*, and there was no leakage in the upper portion of the well, water in the test tube would stand during the test at essentially the same height as before, and result would be negative, merely failing to indicate a possibility that really existed. If there was lateral leakage through the upper strata as well as through *A*, neither alone being quite competent to dispose of the flow from *B*, introduction of the test pipe would cut off the upper leakage, leaving *A* unable to dispose of the entire flow; there would be a rise of water in the tube and possibly a flow. A test of this sort appears to be a true test, because it shows some result, while in reality it is false and misleading. A true test in this case can be made only by placing the packing between the porous beds *A* and *B*.

Where two porous beds, *A* and *B* (Fig. 97), have been traversed, packing placed between, then (1) if *A* equals *B* in productive capacity, water will stand

at same height within and without the test pipe if there is no leakage in the upper beds. (2) If failure to flow was due to such leakage, then a flow will result from *B*, but the additional flow which might be secured from *A* is lost. (3) If *A* has a greater head than *B*, and if there is no loss above, water in the test pipe will actually be lower than that outside, as in Fig. 98. This may be said to be an inverted test and is less misleading than the false and negative test since it plainly indicates an error of manipulation. (4) If, however, there is in this case considerable lateral waste in the upper strata, valuable flow from *A* will be lost just as before the test was made, while *B* may give a rise in the tube, or even a flow, which would foster the impression that a fair test had been made, while in reality the greater flow has been lost. (5) If *A* gives a feebler flow than *B*, but has an equal head, the test will fail of being completely satisfactory only in excluding the feebler flow from *A*. (6) If *A* has a lower head and is a possible means of escape for flow from *B*, then the packing has been placed at the right point and the test gives best results.

In another case let *A* and *B* represent porous beds, lower of which is so conditioned as to drain upper by virtue of a lower outcrop. (1) If drainage loss below is not complete, and if packing is placed above *A* (Fig. 99, Well I), result will be negative if there is no leakage in upper strata. (2) Should there be con-

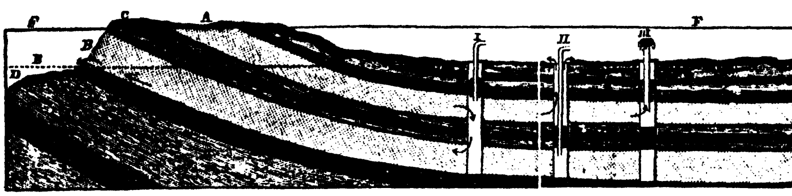


FIG. 99.—Section of strata, showing one correct and two erroneous tests.

siderable loss there, it will be cut off by the tube and packing, and some rise in the tube will be the result in most cases. In either instance result is misleading, particularly in the latter, because the small rise of water is apt to allay any suspicion as to effectiveness of test; flow from the productive stratum is mainly lost below. (3) Suppose packing between *A* and *B* (Fig. 99, Well II): it will shut off flow from *A*, while water in *B*, because of lower outlet, will fail to flow. If there is opportunity for lateral leakage in the upper strata, water from *A* will rise in the well outside the test pipe and pass off into these open upper beds. (4) But if no such opportunity is afforded, it may rise to the surface and overflow outside the test pipe, while water within the test pipe will probably be found lower than before the test was made. Proper method of testing wells known or suspected to present these conditions is to sink a simple seed bag or other obstruction to a point in the impervious stratum between *A* and *B*, which, when it tightens, will shut off flow below. Then a tube with packing sunk at a point above *A* will effectually cut off all leakage in the upper strata, and full capacity of *A* will be tested. Test each water-bearing stratum as encountered, or else vary the final tests so as effectually to exclude all liabilities to error.[25]

Measurement of Depth.* *Cable Measurements.* With a standard outfit, depth is usually measured by the cable as the tools are lowered. Just as top of tools is about to enter the hole, a string is tied to the cable at the bull-wheel shaft; the tools are lowered until this string has gone up over the crown pulley and down to the well head, then another string is tied to the cable at

* See second footnote, p. 223.

the bull-wheel shaft, and so on until the tools reach the bottom. Number of strings tied to cable multiplied by distance from bull-wheel shaft up over crown pulley and down to well head, plus last fraction of this distance, gives total depth of well. Errors may be caused by slipping of strings, by stretching of cable, or by miscount.

Tape Measurement. Measurement with a steel tape has often been found difficult, on account of magnetized condition of casing, produced by jarring of the drilling tools. To avoid difficulties of magnetization, a copper wire or tape may be used.

*Depth Measured Electrically,** (J. G. Thorne).[26] A 2-ft. length of ⅜-in. iron pipe was strung as a weight on the end of a 14-gage, rubber-insulated, copper wire, and securely fastened, the lower end of the wire and of the pipe being about even. This wire ran to the first terminal of a fuse block. Another wire ran from the second terminal to the well casing. From the third and fourth terminals connections were made to a 110-volt circuit. A voltmeter connected to the first and second terminals registered when the iron pipe touching the water surface completed the circuit.

MAINTENANCE OF WELLS

Life of wells depends on stability and reliability of water table, on durability of both casing and strainer, on clogging tendencies of the stratum, and on quality of supply.

Lost Supplies. When a deep well is first sunk, it usually gives copious supplies, but as time elapses, the yield decreases to a small fraction of the original; this is commonly attributed to a decrease in the general supply of the region or the drawing off of the water by better wells. In many wells this is the real cause, but in others, failure is due to: (1) cave-ins at bottom, choking the pipe or damaging the screen; (2) outward leakage through joints or holes corroded in the casing; (3) clogging of the screen or the tributary stratum. This last is the most common trouble. Clogging may result from excessive drafts which fill the perforations in the screen and the voids in the pervious stratum with sand and silt. Clogging of metal screens may result from depositions of iron or lime. Where waters carry acids, the screen is often corroded, the iron set free being redeposited in the sand about the screen, forming impervious coatings. Artesian wells at Savannah, Ga., originally flowed under static heads of 30 to 35 ft., but within 3 years these decreased by 20 to 25 ft.[21] Recession of water-table caused successive abandonment every few years of wells at Bloomington, Ill.[22] Wells may be lost by pulling casing and screen incautiously for cleaning,[27] or by flood, as at San Diego.[28]

Rate of pumping is important; if excessive, either sand may be drawn into the wells or water-table may be permanently lowered, with consequent impairment of yield. The practicable rate in granular materials is directly proportional to the water surface depression incident to pumping.

Where several wells are available, they should be pumped uniformly; if some are left idle, and pumping is restricted to one well, clogging ensues.

* See also *E. N.*, Nov. 25, 1918, p. 1037; *E. N. R.*, Sept. 12, 1923, p. 445; *E. N. R.*, Feb. 8, 1923, p. 266; *J. A. W. W. A.*, Vol. 11, 1924, p. 840.

Operation of the group at uniform rate minimizes clogging; for peak load utilize a reservoir supply.[23] Some supplies have a pump to each well to afford a closer control of the depression. Eight wells widely scattered over the gathering ground at Hibbing, Minn. (9 sq. mi.), were pumped intermittently in regular rotation to maintain the ground-water level.[44] See diagrams in Board of Water Supply *Report* on Long Island Sources, 1912.

Table 60. Wells at Camp Grant. Depression and Specific Capacity[43]

Well number	Pumping rate in g.p.m.	Draw-down below static water level, ft.	Specific capacity, g.p.m. per ft. of draw-down
1	311	13.0	23.9
3	225	10.0	22.5
4	299	8.1	36.8
5	263	10.75	24.5
6	286	6.42	44.5
7	....	....	18.0

Sand pulled into large open wells by pumping causes trouble, particularly in India, where remedy is larger wells, keeping down velocities. Critical head for Indian sands is fixed at 4 to 6 ft. For formulas for determining the rate of pumping, see *Eng.*, July 20, 1920, p. 153, and for methods of testing yield, see *Ibid.* Aug. 27, 1920, p. 273.

Remedies. When matter collected about a well is soft and loose, it can often be removed by pumping heavily into the well. Back blowing with air

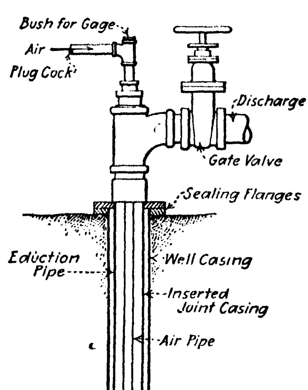


FIG. 100.—Arrangement of piping for back-blowing a well.

is often applied to float out the finer sand clogging the screen. This requires air-lift equipment (see Fig. 100). When both fail, the usual remedy is to pull the casing and screen, or the pump and well point, and replace.

Table 61. Effects of Ground-water Pumping in Diminishing Stream Flow, Brooklyn Watershed,* from 1873 to 1899, by 5-yr. Periods†

(Compiled by L. B. Ward)

Period	Average annual rainfall			Area of watershed, sq. mi.	Driven well supply	
	Total, in.	Collected‡			Expressed as rainfall, in.	Gals. daily per sq. mi.
		%	In.			
1873–1877	43.3	25.1	10.8	52.3	Begun in 1883	
1878–1882	41.6	29.6	12.3	55.1		
1883–1887	43.3	31.6	13.7	64.4	2.9	140,000
1889–1893	45.1	38.4	17.3	65.5	5.8	280,000
1895–1899	43.1	36.3	15.7	66.4	7.8	370,000

Period	Other pumped supplies		Total supply from all sources on watershed (gals. daily per sq. mi.)	Water collected as stream flow, referred to 50 sq. mi. of watershed		
	Rainfall, in.	Gals. daily per sq. mi.		Gals. daily per sq. mi.	Expressed as rainfall	
					In.	% of total
1873–1877	0.2	9,000	517,000	532,000	11.2	25.8
1878–1882	1.0	47,000	586,000	594,000	12.5	30.0
1883–1887	2.3	109,000	652,000	518,000	10.9	25.1
1889–1893	4.2	199,000	824,000	455,000	9.6	21.2
1895–1899	2.7	130,000	746,000	327,000	6.9	16.0

* See later data in Table 6, *Report* on Long Island Sources, Board of Water Supply, New York (1912).
† For methods of measuring rate of underflow electrically, see *J. A. W. W. A.*, Vol. 4, 1917, p. 192.
‡ Referred to watershed as a whole.

Interference of wells means that wells are placed too close together, so that they cannot draw from their full cone area.§ Decreased yield and a smaller return on the investment result. Distance between wells depends on size of pipe, capacity of aquifer and artesian pressure (see p. 240).

Interference of wells has a legal phase. American courts have held each landowner entitled to a reasonable use on his own overlying land in conjunction with equal and correlative use by other landowners. Diversion by a municipality may, therefore, be enjoined on proof of damage. This phase requires study for a new development.[29]

Interference by Salt Water.‖ Some supplies pumped from wells near the seashore have been seriously affected by salt water, although the normal course of underground water is from the land toward the sea. At Spring Creek station, Brooklyn Water Supply, the ground waters had become so exhausted by 1897 that there were 300 p.p.m. of chlorine against a normal of 5. Plant was shut down. One well contained 1400 parts. Jamaica Bay, about $1\frac{1}{4}$ mi. from the wells, contains 10,000 to 15,000 parts. Increase of chlorine at Shetucket wells, L. I., was as follows:

§ See "The Depletion of Ground-water Supplies," by R. E. Horton, *J. A. W. W. A.*, Vol. 7, 1920, p. 167.
‖ See also "Wells Damaged by Sea," by W. P. Mason, *J. A. W. W. A.*, Vol. 8, 1921, p. 59; and "Relation of Sea Water to Ground Water Along Coast," by J. S. Brown, *American J. Science*, Vol. 4, 1922, p. 274.

Date	Chlorine, Parts per Million	Pumping Rate, Mgd.
Oct. 1896* to Mar., 1898.	5	3.7
End of 1898...........	74	6 for 1 month; then 3.5
Oct. 1899...............	246	2.0
End of 1902............	500	1.25

* In 1896 twelve 8-in. wells were put down 175 ft. deep.

Experience shows that several months and sometimes years elapse between beginning of heavy pumping and appearance of chlorine. It is possible that salt water, being heavier, forces out the fresh water for the full depth of the saturated stratum, and sea water gradually advances toward the wells as the fresh water is withdrawn. Capillary action will hold much fresh water in interstices of the gravel, which will dilute the salt water enough to cause the difference between the sea and spring water shown in the analyses; this accounts for the gradual increase in chlorine. After pumping has exhausted fresh water, it would be necessary to close stations several months, if not years, to give the fresh water opportunity to drive out the salt water between the station and tide water. It is, therefore, impractical to continue operating a station after chlorine has passed the danger mark. Liverpool, England, and Galveston, Tex., prosecuted pumping beyond the margin of safety, and sea water in damaging quantities was drawn in.[30a] Metcalf reports cases of water approaching a saline content between 300 and 400 p.p.m., without causing complaints. Beyond this limit, dilution was practiced. In Tampa, the limit of permissible salinity is 350 p.p.m. McFardland[30b] reports a Tampa well in use 15 years, which had originally a salinity of 20 p.p.m; interim values as high as 600, and succeeding values below 100. There seemed to be no regularity to the curve of alkalinity.

Corrosion of Casings. Life of casing cannot be definitely predicted, rate of decay depending on conditions in each well. Casing withdrawn from some wells, 15 to 20 years old, has been found in fairly good condition except at joints, though, as a rule, at this age it is too badly corroded to be withdrawn at all. In Victoria, Australia, the iron lining of a bore was eaten away so as to destroy its continuity in 18 months. There was a saline constituent in the water.[33] At Mankato, Minn., steel casing lasts 12 to 15 years; then it must be replaced by an interior casing of a smaller size. This disadvantage is met by casing with cast-iron pipe† (see p. 234), using brass couplings. A life of 50 years is anticipated.[34] Steel casings at Savannah failed after 26 years.[35] To preserve steel casing and prevent the pollution possible with leaky casings, Kirchoffer has grouted wells at Whitewater, Wis., and elsewhere.[37] Stovepipe casing is sometimes as durable as others; experience in southern California has been that a stovepipe casing, well below surface of saturation, in water containing little oxygen or carbon dioxide, suffers no appreciable deterioration. Wrought-iron casings have been taken from the ground as good as new after 20 years' service. At the ground line, corrosion is the greatest, and light casing does not last more than 10 or 15 years.

Detection of leaks is somewhat difficult. In some wells, water may be heard trickling in or seen by a light ray projected by a mirror when the pump is withdrawn. The admixture of water from outside may sometimes be detected by a difference of hardness, by taste, or by cloudiness due to silt.

† Hardness of water in deep wells would inhibit corrosion; see p. 810.

The remedy is usually to pull the old casing and replace it by new. The time the pipe is allowed to remain before replacement is determined by estimate based on the action of water on the pump tubes or other pipes. An alternative treatment, when the leak is near the surface, is to set a packer in the space between the bottom of the pump tube and casing and fill space above with cement. At Galva, Ill., wells have 12-in. casing for 110 ft., 9-in. casing below. Pump cylinders are 300 ft. below surface. When there are indications of a leak, the pump cylinders are taken out, and a cluster of three electric lights lowered into well, with a shade above. Progress in lowering is followed by aid of a field glass, and leak located.[38]

WELL PUMPS AND THEIR OPERATION*

General Considerations. Water can be drawn from shallow wells through suction pipes attached to pumps situated above ground or in pump pits. Water can be drawn from deep wells only by starting the column of water upward by some sort of propeller located at its lower end, or by air lift. Deep-well pumps have lower efficiency than surface pumps. The cost of a well, which increases rapidly with its diameter, must be balanced against the more efficient operation of a larger pump. Pumping equipment decisions are often wisely deferred until the sinking of the well has indicated the quantity of water available, its depth, and the fluctuations of water level. If a well is out of pumb, a deep-well pump cannot be operated successfully; the air-lift can be substituted. Well water contains air, which will cause trouble when pumped unless provisions for escape are made. Piping from the wells should be on an upgrade to the suction well, to release the air.

Power for Pumps. Direct-acting steam pumps are least economical. Plunger pumps, piston-and-cylinder pumps, and various kinds of centrifugal pumps, are operated by a power head located at the surface, driven by steam, electricity, or internal combustion engines. Compressors for air-lift pumps may be similarly operated; Diesel compressors have proved economical at Fond du Lac, Wis.[64]

Plunger Pumps. Sucker rods extending from the well head move a plunger or series of plungers up and down inside a cylinder supported within the casing, admitting water on the downstroke and propelling it upward on the upstroke. Two or three plungers can operate one above another in the same cylinder, the rods being hollow and working one within another, giving a more continuous discharge than a single-plunger pump. The Luitweiler and Pomona pumps (see p. 501), secure continuous discharge from a double plunger by a cam arrangement. All plunger pumps have limited capacity and high maintenance charges for underground parts.[41] Troubles arise from drop pipe, cylinder, foot valve, plungers, and valves, but more frequently from breaking of rods. This trouble is a minimum in single-stroke pumps of low speed with wooden rods.[41] This type has two merits: a large discharge is obtained from 10-in. or smaller well casings, and speed required is not dependent on lift.[7c] Power head is on a sliding base, which facilitates removal

* (See also Pumping Engines, Chap. 22) Some manufacturers are: American Well Works Co., Aurora, Ill.; A. D. Cook, Inc, Lawrenceburg, Ind; Keystone Driller Co. (Downie Pumps), Beaver Falls Pa.; Layne & Bowler, Memphis, Tenn.; Geo. E Dow Pumping Engine Co, San Francisco; Cal.; Westco-Chippewa Pump Co., Davenport, Ia; Worthington Pump & Machinery Corp.

to one side so that underground equipment may be raised. Barrel must be located at lowest water level expected, otherwise there will be danger of pumping air. Varying water level materially affects the efficiency.[66]

Centrifugal* or turbine pumps are horizontal or vertical and single- or multi-stage. Only vertical pumps are used in small diameter wells; they have submerged rotors driven by power heads through long shafts stayed against vibrations. Horizontal pumps are used in large shallow wells or at the surface for large quantities of water,[42] where the head is not too great. Vertical pumps are common in deep-well practice, see p. 484. The pump in the early Memphis installation,[68] is similar to a screw conveyor, the water being forced up the incline. Centrifugal pumps have reasonable simplicity and durability of underground parts, and are economical of fuel. Water can be raised above the surface by adding another unit at top of shaft, driven by same motor.[41]

Turbine centrifugals are used in wells whose diameters to standing water are 12 in. and larger, and where depth to water exceeds 50 ft. Their problem has been that of vertical shaft bearings; both roller-thrust and hydraulic bearings have been used. A seal placed above the pump converts the well casing into a discharge pipe. If the vertical shaft is enclosed in oil tubing, it is long-lived and free from troubles.[7c] Efficiencies shown in early tests were low. In May, 1915, a new five-stage, belt-connected pump showed a combined efficiency for pump and motor of 58.2 per cent. Good performance is possible for medium-sized pumps. Vertical turbine pumps cost more than pit pumps, and only where a heavy draw-down is required, will they replace them on lifts of less than 100 ft. The field for the pitless turbine is 75 to 250 ft. lift. It is best adapted to developing new wells.[7c]

Advantages of Turbine Pumps.

1. All machinery except the turbine is above ground.
2. Sand and rock cuttings can be handled, as there are no packings or leather cups to be cut out.
3. Moderate first cost.
4. Small size, weight and floor space.
5. Easy and simple to operate.
6. It starts rapidly.
7. Operates at a moderate speed.
8. Has a constant and large flow of water.
9. It is not necessary to prime.

Disadvantages.

1. Very low durability.
2. It is not very flexible.
3. It operates against a limited pressure variation.
4. Only a moderate mechanical pump efficiency.
5. Troubles with bearings.

Electric Driving. Do not use oversize motor; a poor factor results (see p. 523 for electric pumping).

Pump Efficiency. Turbine pumps and motors were purchased for Urbana, Ill., on a guarantee of an efficiency overall of 55 per cent. Tested to 59.3,

* See also p. 482.

delivering 587 g.p.m. against head of 136.9 ft. For methods of testing, see *E. N.* Oct. 8, 1914, p. 720. Centrifugal turbines with 125-hp. motors at Aurora, Ill., gave an overall efficiency of 61 per cent. when raising 630 g.p.m. against a total head of 497 ft. at 1172 r.p.m.[45]

Pump houses should be built over the machinery. Tower of requisite height centered over each well should be provided to aid withdrawal of underground equipment. Towers may be bare steel frames, or architectural structures.[47]

AIR-LIFT PUMP*

Definition. The air lift, or air-lift pump, is an apparatus for raising water from wells by means of compressed air forced to relatively great depth in the well, through a pipe, and so discharged as to mix with the water in small or large bubbles.

Principle of Air Lift. As compressed air enters discharge pipe near bottom at pressure only slightly above hydrostatic head, column of water above is forced upward; mixture of air with water lessens its specific gravity and aids upward motion. Air continues to enter, taking place of rising body of water, until water flows from discharge opening. The moment that part of the rising water is discharged, weight of column becomes less, and air beneath will correspondingly expand, thus reducing pressure on water in discharge pipe below air inlet. Weight of water in well outside of discharge pipe then forces water into discharge pipe, stopping inflow of air. Pressure in air-supply pipe is quickly renewed by its connection with supply, so that it again forces entrance into discharge pipe. This process is repeated until whole discharge pipe, above air inlet, is filled with alternate bodies of air and water, the combined weight of which is enough less than water in well to keep up constant flow of water into discharge pipe. Air issues at mouth at atmospheric pressure, expanding as each succeeding layer of water above it is discharged. Principal loss seems to be slipping back of water layers due to friction of pipe.

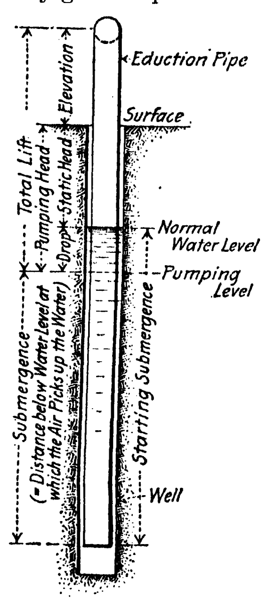


FIG. 101.—Nomenclature of air-lift pump.

Use. Air-lift pumps are used in wells, in mine drainage,† and in industrial plants.

Terminology[48] (Fig. 101). (1) Static head: Normal water level when not pumping, measured from the surface or top of well casing. (2) Drop: Difference between static head and water level when pumping. (3) Pumping head: Level of water when pumping as compared to ground surface or top of well casing. Static head plus drop, equals pumping head. (4) Elevation: Level above the ground surface or top of well casing to which water is raised. (5) Lift: Distance water is elevated from level when pumping to point of

* A partial list of firms which install air lifts: Indiana Air Pump Co., Indianapolis; Ingersoll-Rand Co., New York; Sullivan Machinery Co., Chicago. Talbot Air Lift Co., Philadelphia; Pennsylvania Compressor Co., New York.

† See *Eng. Mining J.*, June 5, 1920, p. 1263.

discharge. Static head plus drop equals lift when discharging at surface. Elevation plus static head plus drop equals lift when discharging above the surface. (6) Submergence: Distance below the pumping head at which the air picks up the water. (7) 100 per cent.: Vertical distance the air travels with the water from the point introduced to the point of discharge. Lift plus submergence equals 100 per cent. (8) Starting submergence: Distance below the static head at which the air picks up the water and includes drop plus submergence.

Types of Air Lift[48] (Fig. 104). (1) Pohlé system* consists of an air pipe carried down outside of the eduction, or discharge, pipe, into which it is

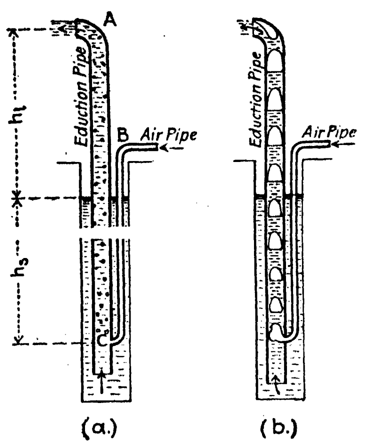


FIG. 102.—Comparison of Frizell (*a*) and Pohlé (*b*) systems of operation; h_s is greater than h_l.

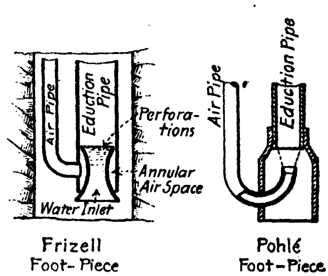


FIG. 103.—Foot-pieces.

either tapped through a short-radius elbow a short distance from the bottom, or turned up into the bottom. It is applicable to lifts greater than 25 ft. It makes use of alternate piston-like layers of water and of air formed in the discharge, or eduction, pipe. With intelligently designed air and water pipes and a plant skilfully adapted to the given wells, economy is claimed as compared with many other systems of pumping. (2) Central system. Air is carried down in a pipe suspended inside of the discharge, the water traveling up around the air pipe. (3) Reservoir system. Consists of an eduction pipe suspended in a casing allowing the air to pass down between the two and mix with the water at the bottom of the discharge pipe.

Frizell system attributes action of air lift to aeration of water in discharge pipe (intimate commingling of air with water), which may be sufficient for moderate lifts, that is, for cases in which the water rises nearly to the top of the well.

Weber† pump utilizes compressed air, but is really a displacement pump.

*Plants with lifts up to 1500 ft. are said to be in successful operation.
†Weber Subterranean Pump Co., N. Y. (Firm dissolved, 1926.)

Table 62. Pohlé Side-inlet Air Lift: Pipe Sizes and Capacities

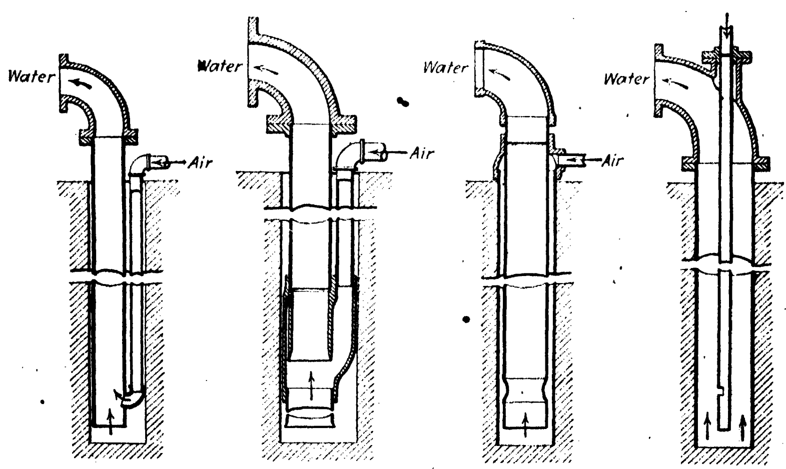
Air pipe connection, in.	Water pipe, in.	Size well, in.	Maximum economical capacity on moderate lift, gal. per min.
$\frac{1}{2}$	1	3	7
$\frac{3}{4}$	$1\frac{1}{2}$	4	20
1	2	$4\frac{1}{2}$	35
1	$2\frac{1}{2}$	5	60
$1\frac{1}{4}$	3	6	90
$1\frac{1}{2}$	$3\frac{1}{2}$	7	120
$1\frac{1}{2}$	4	8	160
$1\frac{1}{2}$	5	9	250
2	6	10	350

Table 63. Pohlé Annular Foot-piece Air Lift: Pipe Sizes and Capacities

Well, in.	Pipe sizes			Max. economical capacity, moderate lift, gal. per min.	Dimensions of foot-piece	
	Disch., in.	Air, in.	Tail, in.		Outside diam., in.	Length, in.
4	$1\frac{1}{2}$	$\frac{1}{2}$	2	20	$3\frac{11}{16}$	11
5	2	$\frac{3}{4}$	$2\frac{1}{2}$	35	$4\frac{13}{32}$	12
6	$2\frac{1}{2}$	1	3	60	$5\frac{1}{2}$	$12\frac{1}{2}$
6	3	1	$3\frac{1}{2}$	100	$5\frac{7}{8}$	13
6	$3\frac{1}{2}$	$1\frac{1}{4}$	4	140	$5\frac{15}{16}$	13
8	4	$1\frac{1}{2}$	$4\frac{1}{2}$	190	$7\frac{1}{8}$	$13\frac{1}{2}$
8	$4\frac{1}{2}$	$1\frac{1}{2}$	5	225	$7\frac{9}{16}$	$13\frac{1}{2}$
10	5	$1\frac{1}{2}$	6	300	$9\frac{3}{8}$	14

NOTE.—"Maximum Economical Capacity" is based on 60 per cent. submergence and discharge of 12 gal. of water per min. per sq. in. area of discharge pipe in the smaller diameters, and 15 gal. per min. per sq. in. in the larger sizes.

Submergence. For most effective operation, use the ratios of Table 65. These apply to working conditions, after heavy pumping has lowered the water-table. Before accepting a well, the testers should ascertain this



Pohlé Side Inlet. Pohlé Annular Foot-Piece. Saunders (or Reservoir) System. Central Air Pipe.

FIG. 104.—Types of piping.

minimum level. Special plants are working satisfactorily with submergence less than 30 per cent. and some with over 75 per cent. Submergence governs starting and working air pressures required.

Lorenz's Theory.* The formulas take account of losses of energy occasioned by slip, pipe friction, etc., and are of practical use in designing pumps, provided the necessary experimental coefficients are known.

During operation of pump, the following equations of heads hold between point c (Fig. 102a, p. 250) in pump and a point at same elevation outside:

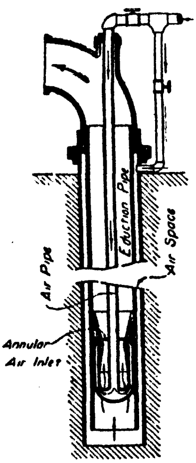


Fig. 105.—Combination air lift.

$$h_s - \frac{p_i - p_b}{u_w} = \frac{v_i^2}{2g}(1 + c_e) \qquad (1)$$

$$\frac{q_b p_b}{q_w u_w}\log_e \frac{p_i}{p_b} = h_l + \left[\frac{\left(1 + c_p\frac{l}{d}\right)(q_b + q_w)^2 + c_e q_w^2}{2g a_p^2}\right] \qquad (2)$$

$$\frac{1}{q_w}\frac{p_b}{u_w}\log_e \frac{p_i}{p_b} = \frac{1 + c_p\frac{l}{d}}{2g a_p^2}(q_b + q_w) \qquad (3)$$

$$\left(1 + c_p\frac{l}{d}\right)(q_b^2 + q_w^2) = 2g h_l a_p^2 + c_e q_w^2 \qquad (4)$$

a_p = area of eduction pipe in sq. ft. c_e = coefficient of entrance. c_p = coefficient of pipe friction and average slip. h_l = lift, ft. h_s = depth of submergence, ft. l = work output, ft.-gal. per sec. p_b = barometric pressure, acting on the surface of the water in well and also on the discharge end of pipe. d = diam. of eduction pipe, ft. p_i = absolute pressure at inlet in foot-piece. q_b = discharge of air at pressure p_b. q_w = discharge of water, cu. ft. per sec. u_w = density of fluid pumped. v_i = velocity of the liquid in the eduction pipe below the air inlet.

If the maximum discharge, determined from the capacity of the well, and the area a_p of discharge pipe, determined from the diam. of the well, and also the lift and the known coefficients c_e and c_p are given, the volume of free air required may be computed by formulas (3) and (4), from which the submergence h_s can then be computed by equation (1). Equation (2) then gives the relations between any desired values of q_b and q_w using the same pressure p_i.

Ingersoll-Rand Formula.† Ratio of volume of air to water raised:

$$V_a = \frac{h}{C \log \frac{H + 34}{34}}$$

V_a = cu. ft. free air per min. actually required to raise 1 gal. of water; h = total lift in ft.; H = running submergence in ft.; C = constant as in Table 64.

Table 64. Constants in Air-lift Formula

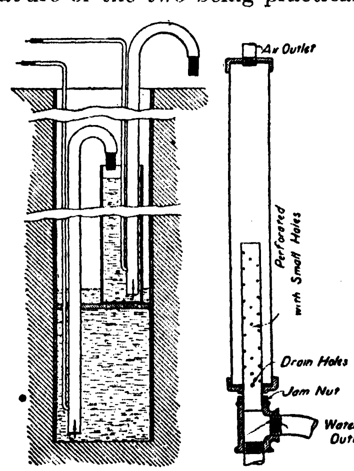
Submergence, per cent..	75	70	65	60	55	50	45	40	35
Constant C............	366	358	348	335	318	296	272	246	216

* See also *Power*, Nov. 23, 1920, p. 818. The following pages are based on *Bull.* No. 450, October, 1911, University of Wisconsin, Davis & Weidner, unless otherwise credited.

Table 65. Showing Customary Allowable and Best Submergences
(Ingersoll-Rand Co.)

Lift in feet	Customary allowable percentage submergence	Best percentage submergence	Single stage or compound air compressors
20	55 to 70	(65-70)	Single
30	55 to 70	(65-70)	Single
40	50 to 70	(65-70)	Single
50	50 to 70	(65-70)	Single
60	50 to 70	(65-70)	Single
80	50 to 70	(65-70)	Single
100	45 to 70	(65-70)	Single
125	45 to 65	(65)	Single
150	40 to 65	(60-65)	Single
175	40 to 60	(55-60)	Single
200	40 to 60	(55-60)	Compound
250	40 to 60	(55-60)	Compound
300	37 to 55	(50-55)	Compound
350	37 to 55	(50-55)	Compound
400	37 to 50	(45-50)	Compound
450	35 to 45	(40-45)	Compound
500	35 to 45	(40-45)	Compound
550	35 to 45	(40-45)	Compound
600	35 to 45	(40-45)	Compound
650	35 to 45	(40-45)	Compound
700	35 to 40	(40)	Compound

Return Air Pump. In rising through eduction pipe there is a transfer of heat between air and water; the temperature of the two being practically equal at point of discharge. Therefore, when pumping from underground supplies, air from discharge pipe will be cooler than atmosphere during warm months. For each 5° fall in temperature of free air entering compressor, a saving of 1 per cent. in energy in compression may be effected. Hence, where wells are close to power house, economy may be effected by connecting inlet of compressor with top of well-casing head. A separator for this purpose consists of a cylindrical drum about 18 in. in diam., 8 or 10 ft. long, attached to casing head, as in Fig. 106.



Multiple Air Lift. Air Separator.
FIG. 106.

Disadvantages of Air-lift Pump. (1) Low efficiency. Generally credited with only 25 to 33 per cent., but notwithstanding low efficiency of pump itself, entire plant in some cases develops a duty which compares favorably with other systems. Variation in submergence ratio due to lowered water level reduces efficiency. (2) Great depth of submergence, see Table 65. A single air-lift pump cannot be used in a shallow well or reservoir, except to raise liquid a small distance, owing to high percentage of total length of pump which must be submerged to give good efficiencies. This limits the air lift principally to deep-well pumping. Multiple-stage pump overcomes difficulty,

but probably at reduced efficiency. (3) Limited horizontal pumping. Several plants have been installed to pump a considerable horizontal distance, but such plants are not considered efficient. Bissell[41] claims that secondary pumping is often required to raise water above surface. A booster at Sharpsburg, Pa., forces discharge through 1350 ft. horizontally. Efficiency is not stated.[50] The air in passing through a horizontal or even an inclined pipe, is not likely to be evenly distributed throughout cross-section, but to pass along upper side, allowing a large space in lower portion for water to slip back past the bubble. In a horizontal pipe, air cannot exert any buoyant effort to aid in discharging water, and its expansive force, which might be used in overcoming pipe friction, is not likely to be effective on account of serious slip. Where it is desired to convey water to a point distant horizontally from the well, eduction pipe should be carried vertically to a height equal to friction head in the horizontal conductor, and at its top fitted with an air separator. (4) Aeration. The thorough aeration of water pumped is generally regarded as an advantage, but under some circumstances it promotes rusting and consequent destruction of eduction pipe, and in some cases causes a deposit of salts which clogs passages, especially in foot piece. Opinion has also been expressed that compressed air causes an excessive growth of algae; bacterial content of water is somewhat increased by air lift unless air supply is filtered. (5) First cost is moderately high. (6) Flow is intermittent. (7) Lack of flexibility to meet variations in demand and less economy in operation[67] led to replacement by turbine pumps at St. Petersburg, Fla.

Advantages of Air-lift Pump. (1) Large capacity. When conditions are suitable, an air-lift pump will discharge more liquid from a well of small bore (4 to 10 in.), than any other type, due to fact that almost the entire cross-sectional area of the well is available for flow of liquid, and action is nearly continuous. The air lift affords a means for testing the capacity of a well even if it is not to be permanently installed. (2) Low maintenance cost. Owing to simplicity, cost of maintenance is very low; life of the pump is almost indefinite. Sometimes air pipes and foot piece become clogged with oil carried over from compressor cylinders and have to be removed and cleaned; this rarely occurs, and cost is small compared with cost of replacing a mechanical pump. Absence of moving parts in well makes pump especially fitted for dirty water, sewage, mine water, acid or alkaline solutions in chemical or metallurgical works, or other corrosive liquids. Liquids that attack metals, such as brine, sulphuric acid, etc., may be pumped by air lift, because pump and appurtenances may be replaced at small expense and loss of time. Air lift as a dredge pump has been successfully, but not extensively, used. (3) Low operating cost. Where wells are scattered, or remote from power house, air lift has advantage over steam-driven pump. In a deep well pump driven by steam, each well must be equipped with a separate engine and working barrel, which entails heavy condensation losses through long steam supply pipes; expense of attendance is great. In air-lift pump, transmission loss is much smaller; no attendance at well is required, operation being controlled by a valve in the power house. (4) Not affected by high temperatures. Fluids of different densities and temperatures may be handled to advantage where other types of pumps would be prohibited. In a hot liquid, air absorbs

part of heat and is increased in volume, so that discharge for same expenditure of free air is greater with hot than cold liquids. This results in a considerable gain in efficiency. (5) Aeration. Iron is oxidized by aeration and supply is thereby improved.* Aeration is especially advantageous in pumping sewage. (6) Reliability. Air pumps are not liable to sudden stoppages or breakdowns. (7) Permanence of yield. Wells may be readily back blown to free strainers and maintain capacity. (8) Lower temperature of water. (9) System is readily adjustable to new pumping conditions due to lowered water level.[51] (10) Many wells, of various depths, can be operated economically from a central plant.

Use of air lift[52] should be limited to cases where efficiency can be sacrificed for sake of reduction in maintenance expense, increase in well output or increase in reliability, to crooked wells or those in which water must be pumped from a greater depth than 200 ft.

Efficiency[52] of even a well-designed air lift is low; varying from 10 per cent.† for a lift of 600 ft., to 45 per cent.‡ for a lift of 50 ft.; and is greatly influenced by ratio of submergence to lift. Maximum efficiency is secured with submergence 2.25 times lift. Lowering of water level affects it. Reduction in submergence lowers efficiency more rapidly than increase. Plant at Galesburg, Ill., with submergence ratio of 2.2, and lift of 311 ft., gave efficiency of 37.5 per cent.[54] Plant at Fort Bliss, Tex., gave air-lift efficiency of 51.9, and wire to water (overall) efficiency of 28.6 per cent.[55] An efficiency of 59 per cent. is reported for an 8-in. well at Camden, N. J.[56] A plant near New York operating both air-lift and deep-well pumps in 8-in. wells found that air lift had four times the operating cost of deep-well pump.[49] Tests on air-lift and deep-well centrifugals at Mobile gave for wire-to-water efficiency 10.6 for air-lift and 60 per cent. for centrifugals.[73]

Table 66. Effect of Submergence§[52]

Submergence, ft.	Efficiency, per cent.
8.70	26.5
5.46	31.0
3.86	35.0
2.91	36.6
2.25	37.7
1.86	36.8
1.45	34.5
1.19	31.0
0.96	26.5

Table 67. Air-lift Tests to Determine Best Flow from Deep Wells, Hattiesburg, Miss.§[52]

Length of air pipe, ft.	Flow gal. per min.	Lift ft.	Efficiency, per cent.	Duty gals. per hp.-hr.
224	1,495	47.5	32.6	1,630
208	1,459	45.8	31.5	1,640
184	1,419	44.3	31.1	1,670
162	1,359	39.4	30.0	1,790
142	1,299	37.7	30.6	1,920
124	1,219	37.5	32.8	2,070
105	1,106	37.1	36.7	2,340
86	1,008	32.5	33.4	2,450
79	904	29.3	31.3	2,530
67	802	26.0	30.5	2,750
43	690	21.2	29.8	3,240

• Flow about 1,100 gals. per min.
Lift about 37 ft.

* For results at Memphis, see *E. N. R.*, Dec. 18, 1924, p. 992; Public Works, Vol. 54, 1923, p. 371; Public Works, Vol. 55, 1924, p. 13.

† Many old plants test to 10 per cent. due to combined obsolescence and lowered water table.[53]

‡ For pump only, exclusive of compressor losses.

§ "Pumping by Compressed Air," by E. M. Ivens. Speed of air compressor was adjusted so as to keep rate of flow of water constant, while length of air pipe was varied; dimensions of well and air lift were as follows:

Total depth of well, ft.	453.5
Inside diameter of casing, in.	9[illegible]
Inside diameter of air pipe, in.	2[illegible]
Inside diameter eduction pipe, in.	9[illegible]
Static lift, ft.	3 to 4.0

Requirements for Efficient Air Lift.[57] 1. Means to secure a perfect mixture of minute air bubbles and water at the point at which air is injected into water. Then each particular small bubble will start its lifting effect at once. 2. A Ventura, or choke, just above the mixer. This will increase the velocity and give a jet effect at this point. 3. Eduction pipe arranged with proper enlargement, to allow for expansion of air so far as possible and to prevent excessive velocity toward discharge. 4. An absolutely smooth passage for the air and water. Even the swirl caused by the recess in a coupling, occurring as often as it does in a long pipe, will cause much loss. 5. Proper proportioning of air and water pipes.

Tests. *Westinghouse Air Brake Co.*[70] Nearly 1800 experiments, on nearly 400 different combinations of discharge pipe, diam., lift, and submergence, made on an actual well, 6 in. diam. and 174 ft. deep, led to the conclusions: (1) Rate of delivery of water, and air consumption per gallon, with fixed size of discharge pipe, are practically constant for all lifts, provided ratio of lift to submergence is maintained constant. (2) With a discharge pipe of given diameter, delivery decreases and air consumption per gallon increases as ratio of lift to submergence increases. (3) With a fixed ratio of lift to submergence, air consumption per gallon decreases as size of discharge pipe increases. (4) Least air pressure that will give continuous flow is the proper pressure to use. A slightly lower pressure gives intermittent delivery, and the amount is much decreased, though air consumption per gallon is slightly lower than with continuous flow. With pressure higher than required to give continuous flow, delivery is increased somewhat, but air consumption per gallon delivered is increased in greater ratio; and with further increase in air pressure, a point of maximum delivery is reached beyond which delivery is decreased. Sound of discharge is a reliable guide to proper regulation of air supply. (5) It appears from (2) that by increasing submergence, *i.e.*, locating foot-piece deeper in water, for a given lift, air consumption is progressively reduced; but as required air pressure is increased, a cubic foot of air represents greater power. A curve representing variation of horsepower required per gallon of water delivered, with depth varying, shows that the power first decreases with increasing depth, then reaches a minimum, and thence increases. Ratio of lift to submergence at this minimum point may be called "economical ratio." (6) For a given size discharge pipe, the economical ratio decreases as lift increases; *i.e.*, submergence should be increased in greater ratio than lift. For a given lift, economical ratio increases (submergence decreases) as size of discharge pipe increases. (7) A tail piece, or projection of discharge pipe below air inlet, is essential in starting, as it tends to prevent air from backing down into the well and rising in casing outside the discharge pipe. (8) Any jet or pipe introduced into discharge pipe to serve as an air inlet has no value, and is detrimental by forming an obstacle to free passage of water. (9) Size of air pipe is determined only by considerations of friction loss required to force air through pipe.

University of Wisconsin Test of 13 *Air Lifts.*[63] Conclusions from 1911 tests are given in *Bull.* 450. Tests in 1923 by Ward & Kessler led to conclusions:

1. Efficiency of an air-lift pump depends primarily upon conditions of flow in eduction pipe.

2. Great refinement (small air openings for dividing air into fine bubbles, and special devices for mixing air with the water) in design of foot pieces is not necessary. No central nozzle or projecting part should obstruct flow of water in foot piece.

3. In a given well, maximum efficiency results at some particular rate of pumping. The smaller the pump, the narrower the range of rate of pumping in which high efficiencies may be obtained.

4. Per cent. submergence required for maximum possible efficiency generally ranges from 65 to 75. The lower range is approached in wells with high delivery head.

5. Very small pumps give relatively high efficiencies with low submergences. A 1-in. pump shows good efficiency at 45 per cent. submergence.

6. It is possible that the air-lift pump may be satisfactorily adapted to the pumping of small wells, *e.g.*, for rural water supplies.

7. Combined friction and slip losses due to flow in eduction pipes follow a different law than that which governs flow of water, or of air, in a pipe.

8. There is a comparatively simple relation between frictional losses and velocity of flow in an eduction pipe for any particular mixture of air and water.

9. At a given velocity of flow the losses in eduction pipe increase as ratio of volume of air to volume of water increases.

10. There is one velocity of flow, for any ratio of volume of air to volume of water which gives a minimum loss of head. Losses increase very rapidly when average velocity is reduced below this. Rate of increase of losses with increase of velocity depends upon diameter of eduction pipe. Relatively high velocities may be used in large eduction pipes. In small eduction pipes, losses increase rapidly with increase of velocity above the velocity which gives maximum efficiency.

11. Joints in eduction pipes should be smooth. Changes in pipe sizes should be gradual. A sudden enlargement is detrimental to efficient operation. Eduction pipes should be vertical. A horizontal travel of a mixture of air and water results in separation of air and water.

12. In many cases, eduction pipes of uniform diameter could be designed to give better efficiencies than are now obtained with varying diameters. This is particularly true of pumps used in wells in which delivery head is relatively low.

13. Many pumps of varying diameter are designed on the faulty assumption of a straight-line hydraulic gradient from foot piece to point of discharge, which results in use of velocities that are entirely too high in the lower sections of eduction pipe.

14. Loss of head-velocity analysis is most satisfactory method of correlating experience with air-lift pumps. This makes possible the application of experience to the practical design of pumps for conditions different from those encountered in the tests.

15. Many experimental tests of pumps do not include sufficient data for complete analysis of conditions of operation. Temperature of liquid pumped

should be recorded in field tests. Pressure measurements should be taken, when possible, at points in eduction pipes where pipes of different size are joined. These pressures may be obtained at small cost, if ¼-in. gage pipes are installed when eduction pipe is being assembled and lowered into well. Pressure observations then can be made in same way that level of water in well is usually obtained by use of "tell-tale" pipes. There is particular need for further experimental tests on eduction pipes of large diameter.

16. Test models of air-lift pumps less than 40 ft. long are apt to give results different from those obtained with long pumps. Losses which are relatively insignificant in large pumps become important in short, air-lift pumps.

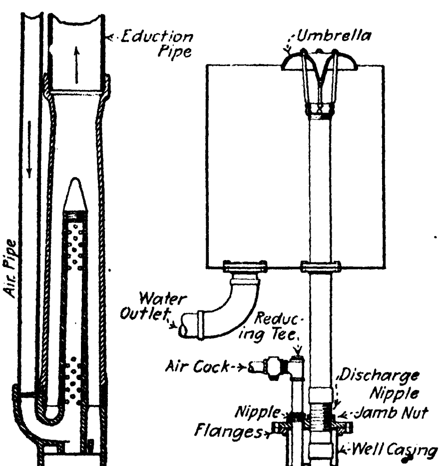


FIG. 107.—Sullivan standard air-lift pump and umbrella well head, with casing flanges and connections.

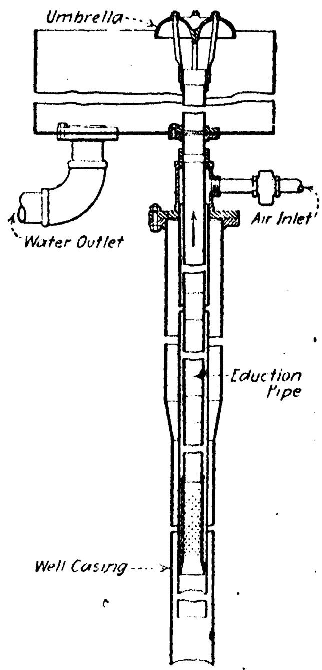


FIG. 108.—Sullivan special air-lift pump and umbrella well head.

Table 68. Central Air Pipe Air Lift: Pipe Sizes and Capacities

Size of casing, in.	Size of air pipe, in.	Capacity, g.p.m.
3½	1¼	80 to 100
4	1½	100 to 150
5	2	150 to 250
6	2	275 to 375
8	2½	500 to 650
10	2½	775 to 1000

Booster System. By this means, water may be lifted to an elevation above the surface by using again the air employed in the air lift proper. This system is especially suitable where the elevation must be accomplished at some distance from the wells. There is an open exhaust for released air to escape to atmosphere, but this is throttled by a valve so as to retain sufficient pressure above the water to force it through discharge pipe to required distance

and elevation. With exhaust pipe full open it will discharge air and water; then the valve is partially closed until only air escapes. Engineman operates complete plant from compressor, no adjustment being required other than varying speed of compressor to secure greater or less amount of water. Any number of wells and "boosters" may discharge into a common delivery pipe.

Bibliography, Chapter 10. Wells

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CHAPTER 11

INFILTRATION GALLERIES

Types. American practice is to construct conduits below the water-table in water-bearing strata bordering streams, so as to collect seepage from the stream, while German practice (p. 264) is to intercept the flow towards the stream. Collecting conduit may be a tunnel, a walled conduit with many openings, or a pipe line with open joints or with inlets similar to the strainers used in filtration, see p. 724. In American practice the conduits generally lead water to a pump well, for elevation to height needed for use. For Smith system at Parkersburg, W. Va., see *T. A. S. C. E.*, Vol. 81, 1917, p. 775.

Galleries vs. Wells. A gallery intercepts water more completely than wells. Under proper topographical conditions, the suction pipe may be eliminated and trouble from pumping air minimized. A gallery supply costs less to investigate than a system of wells. Pumping charges are less, particularly when the well level is at considerable depth. A gallery has lower depreciation, but higher construction cost. Galleries are not advisable except for water-tables of known stability. Many galleries have failed to produce water permanently in sufficient quantities.[1] Siltation or cementing of porous strata surrounding a gallery will curtail the yield, unless means for cleansing (see p. 261), are provided. Wells may be deepened to accommodate the receding water-table, but galleries must be abandoned. Ease of pollution causes sanitarians to look upon the gallery with suspicion.[1] Wells draw deeper and more sterile waters, but the mineral content may be objectionable. Pollution of gallery water at Austin, Tex., was stopped by an intercepting sewer.[2]

Infiltration rates of 0.010 to 0.025 mgd. per acre of area drained by the galleries may be expected from a properly designed infiltration system.[3]

Table 69. Yield of Infiltration Galleries[4]

City	Topography	Geology	Annual rainfall in inches	Catchment area, acres	Average yield of gallery	
					Cfs.	Per cent. of rainfall
Rennes	Gently sloping, broken by small valleys, cultivated	Decomposed granite in place	35.0	8220	9.8	29.5
Brussels	Gently sloping, broken by small ravines, cultivated	Porous sand	27.6	8960	8.3	29.4
Brussels	Same, forested	Same	27.6	2250	2.6	37.0
Liege	Gently sloping	Fissured white chalk	31.0	8780	5.7	18.1
Brookline, Mass.		Gravel	45.0		6 to 7.5	

Collecting pipes are used both separately and in conjunction with collecting galleries. Valves can be placed on them and means thus provided for back blowing to improve capacity. This is an advantage. Perforations in col-

lectors should have a gross area sufficient to limit velocities of inflow to those that will not carry sand into the system.

Clogging. A timber gallery in North Platte River was 3 × 3 ft. inside, made up of 2 × 4's spaced $\frac{5}{8}$ in. apart. The hard water and gravel were so constituted as to produce cementing conditions which greatly diminished the porosity of the gravel. Relief was secured by excavating and removing the incrusted material; but the impervious condition would recur within a few weeks; the gallery was finally abandoned.[5]

Self-cleansing. Prince[5] installed on North Platte River a cast-iron main collector with Ys at each of 10 branches, 36 ft. long, controlled by a valve. Inlets on each branch were 1-in. holes about 10 in. on centers in three lines. By reversing flow from the reservoir, by-passing the pumps, and concentrating all flow on one branch by closing all other valves, a reverse current is set up which removes clogging of influent orifices and breaks up incipient cementing of gravel.

Denver. For years, Citizens Water Company, Denver,[6] secured a supply from about 1 mi. of timber crib, 30 in. square in section, and about 1 mi. of perforated 30-in. pipe, both submerged from 14 to 22 ft. in water-bearing sands of Platte River.* Timber crib was open at bottom, so that, with openings of cribbing, perhaps half its superficial area permitted inflow; for the pipe, the net area of perforations probably bore a much smaller ratio to circumferential area. It may be assumed that in 2 mi. the total area of inlet openings equaled 25 per cent. of the surface exposed to sand, affording a total net area of inlet openings of 26,000 sq. ft. Supply secured was 400,000 to 450,000 cu. ft. per day, equivalent to 13 cu. ft. per day per sq. ft. of inlet opening, or velocity of inflow of 0.00015 ft. per sec. As the water approached radially from every direction it may be assumed that the maximum velocity through the sand was less than one-fourth that through the openings. Assuming an average head of 16 ft. acting on the cribs, the spouting velocity of water entering the pipe from a free body of water would be 32 ft. per sec.; actual average velocity was only $\frac{1}{200{,}000}$ of such rate, corresponding to $V = 0.00004\sqrt{H}$.

Des Moines. Total length of gallery completed and in use at time of test was 8480 ft. Manholes, 600 ft. apart. Depths of galleries 20 to 25 ft., submerged 5 to 10 ft. at low water. In extensive and uniform coarse sand deposits of the Raccoon River Valley. The newer part, 5160 ft. in length, built of concrete rings, has a cross-section of 12.6 sq. ft. Older galleries are rectangular, 15 to 20 sq. ft. in cross-section. When yielding at rate of present maximum pumpage, 14 mgd., aver. vel. at lower end of main gallery is only 0.9 ft. per sec., decreasing progressively to practically zero at far end.[7] Flooding was resorted to for the purpose of increasing the yield in the drought of 1922. See *J. N. E. W. W. A.*, Vol. 38, 1924, p. 203.

Wantagh and Massapequa galleries of the Brooklyn, N. Y., Water Supply, respectively 12,600 and 18,200 ft. long, consist of vitrified sewer pipe laid with open joints in sheeted trenches 10 to 15 ft. below normal ground-water level and surrounded with coarse gravel, over which a layer of fine gravel was

* Many Ohio River towns employ this method. See *T. A. S. C. E.*, Vol. 81, 1917, p. 821.

placed. Sand was used for refilling the trenches. The whole region is a sandy, gravelly morainic outwash. The lower line of sheeting was usually

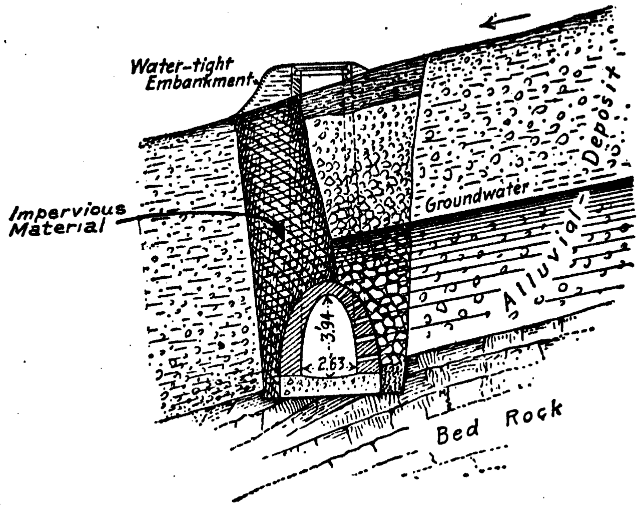

FIG. 109.10a

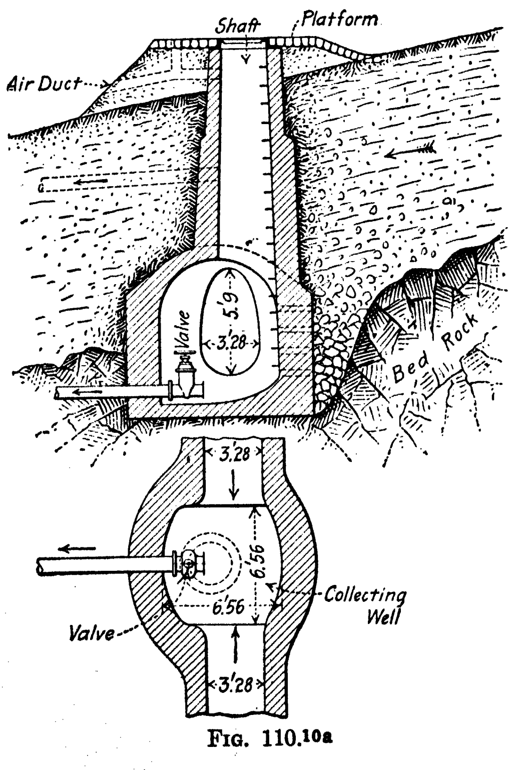

FIG. 110.10a

left in place. Beginning at the pumping station at the central point, each gallery was excavated in both directions, approximately at right angles t

the direction of underground flow. Gradients of the galleries toward their central wells are sufficient to carry double the estimated normal yield of 1,000,000 gal. daily per 1000 ft. Pipe diameters increase from 20 in. at extremities to 36 in. at the pump wells. To facilitate construction and to collect sand which may enter the galleries, manholes with sumps are provided every 250 ft. Crossing a village, Wantagh gallery was laid with cast-iron pipe for 1800 ft. to exclude polluted water. Construction cost proved to be about twice that for driven well systems in same locality, per million gallons daily, but total cost of water, delivered, including interest, sinking fund, taxes,

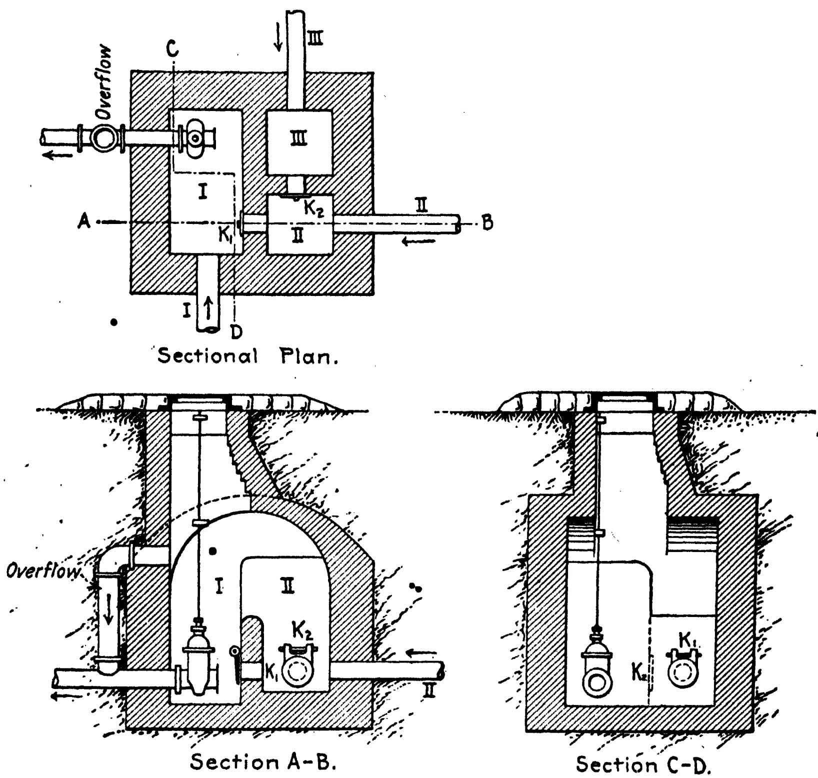

FIG. 111.—Underground collecting reservoir with flap valves.[10a]

operating expenses, extraordinary repairs and depreciation, is much less per million gallon than that of water obtained from the wells on the same watershed. (Wm. W. Brush.)

Subterranean Storage Gallery, Santa Barbara, Cal.[11] Tunnel, 5 ft. × 7 ft., penetrates 2 mi. through sandstone and shale into a hill containing a natural underground reservoir; 2000 ft. from tunnel portal is a masonry bulkhead; through this passes an 8-in. pipe which conveys the underground supply to the city reservoirs during the dry season. A valve outside the bulkhead is closed in autumn when city no longer needs this auxiliary supply; during the winter, the pressure behind the bulkhead rises rapidly. In the spring, about 1 mgd. is available for 3 months. The underground reser-

voir has an estimated capacity of 45 mg. Normal flow from underground is about ½ mgd.

German Methods.[10a] In building an infiltration gallery, clayey material, if possible, is rammed solidly against the wall toward the valley and above the roof so as to form an underground dam which backs up the underground stream, and forces it into the gallery. Such a gallery collects as much water as possible from the ground, and the underground dam utilizes the ground as a collecting and equalizing reservoir. In refilling the trench on the side toward the hill, broken stone or gravel is solely employed, the coarser layers being next to the gallery; on top just below the surface, a layer of impervious

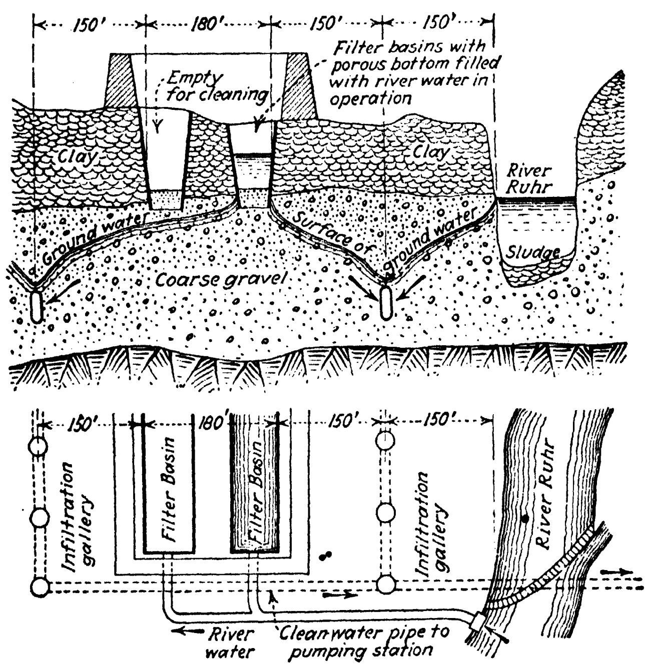

FIG. 112.—Filter basins to augment water supply from infiltration wells and galleries, Ruhr Valley, Germany.[12]

material is placed to prevent surface water from entering. Figure 109 is a cross-section of a collecting gallery (large enough for a man to pass through) the bottom resting on the solid rock; the dotted lines indicate a manhole, which reaches 10 in. above the surface, to prevent earth, etc., from being washed in. According to size of plant, the inspection and ventilation shafts may be large enough for a man to enter, or only 8-in. pipes. Figure 110 represents a collecting reservoir, in communication on both sides with two collecting galleries (large enough for a man to walk through); a delivery pipe leads from this reservoir, and the flow is regulated by a valve.

Subterranean Galleries at Wiesbaden are good examples of another method of utilizing underground water supply. The water-bearing mountain is composed of fissured quartz in the interior, covered with slightly porous

rock. Galleries were driven at the necessary elevation to give sufficient head for supplying water; they are pierced where flows occur, to receive the water. Total length of galleries is 3000 meters, of which 2000 are in the porous covering. Inequalities of flow are harmonized with demands for water by building doors in the galleries. These doors are of wrought iron, and made watertight by rubber gaskets on the frame. Behind these doors, erected in pairs, the surplus water is stored in ample quantities for use during the dry spells. Water is taken off by means of a valve beside the door. The local situation of the doors is determined by existing lateral outflow from the subterranean reservoirs, since the damming by means of the door should not raise the water to such a height as to prevent free discharge from the fissures. The distance to the next doors downstream is fixed partly by the occurrence of soft formations where they can be erected, and according to the existing free flow, the height of which must not be exceeded by the second doors. A succession of these doors in an inclined gallery gives the appearance of a flight of stairs to the water surfaces behind the doors.[10b]

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CHAPTER 12

SPRINGS

Development. Springs may issue from fissures in rock, or from exposed faces of porous strata, or wherever ground water has access to the surface. Development* of a spring supply consists ordinarily in freeing the flow by removal of obstructions, providing a wall to exclude pollution and to form a reservoir, and supplying a cover and overflow for the reservoir. Flows from porous strata can be increased by a series of collecting pipes;[1] or an infiltration gallery (see Chap. 11).

Yield of Springs.[1] The upper Vanne Springs, Paris, France, are fed from an area of 136,300 acres. They supply 24.5 cfs. (16 mgd.) from an average rainfall equal to 28 in. (21 per cent. collected). The largest spring has a minimum yield of 2.5 cfs. and average 7. The largest spring supplying Vienna, Kaiserbrunnen, averages 20 cfs., varying from 8 to 34. The yield of springs is subject to the same conditions as wells; see p. 240. Roanoke, Va., is supplied from springs issuing from the limestone and having a flow varying from 3.0 to 5.0 mgd.†

Classification. (1) Springs which bubble out of fissures of rock layers; (2) those which issue from the weathered rock strata above or beside the bedrock; (3) those which bubble up in alluvial land.

Utilizing Springs in Rock Strata.[3a] First remove all earth and weathered rock over and about the point where the water issues, so as to lay bare the bedrock. Water issuing from the rock is then caught in a small reservoir with stone, concrete or brick walls, whence it is conducted through pipes to the point of consumption.

Earth slopes around a spring should be strengthened by sod or paving to prevent being washed down. Make sure that rain water cannot find its way to the rock against which the spring house is built. Figure 114 shows a spring house without entrance, one with entrance on top, and one with a door at the side, respectively.[3b]

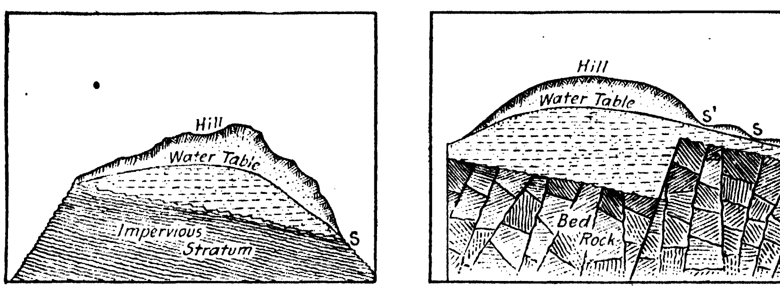
Weathered Rock. Water issuing from partly loose and sandy rock layers often carries much sand. Springs with reservoirs which are not large or deep enough, and into which surface water passes after a rain, are often cloudy on account of their content of suspended matter. Such water should be passed through a sand-catcher, or settling basin, before entering the clear-water tank or reservoir. To keep the water pure, it is necessary to provide for this settling chamber a concrete or brick roof covered with earth, and to keep the air out. An auxiliary conduit through which the water can be passed when the settling chamber is to be cleaned, and means for emptying it, are also necessary. A

* See "Utilizing a spring as source of water supply for a town," *J. N. E. W. W. A.*, vol. 11, 1896, p. 156.

† There is a supplemental supply from other sources.

suitable settling chamber is rectangular, of such a cross-section that water passes through it at a slow speed, say 10 ft. per hr., according to the nature of the suspended matter.

Utilizing Springs in Alluvial Soil.[3b] The method described in the last paragraph applies also to springs in the upper regions of alluvial land, such as mountain valleys. Springs in the lower regions of alluvial land occur mainly in trough-like depressions of large river valleys and in low-ground districts, which are either at a lower level than the ground water in the neighborhood or at place where it is dammed by deficiently pervious or impervious obstructions. On the shores of rivers and lakes, especially in inlets, ground water often issues from the soil as a spring and passes into the river or lake; springs which are covered by water reveal their existence by rising air bubbles as well as by small sand runs. Springs in low ground always rise from below, and often make themselves manifest by bubbling, so that they are comprised within the class of bubbling springs. Springs in the mountain valleys rarely behave in this way; when they issue from the soil under pressure from a higher level, they may be called fountain springs. Bubbling springs in low ground,



Stratum Spring. Fault Spring

FIG. 113.—Two formations causing springs.

on account of the slowness with which the water flows off, often cause a considerable accumulation of water in the form of a pool, swamp, or pond, covered with plants and containing animal life. When it is possible to drain the pool artificially, the surroundings of the springs are carefully cleaned to remove mud, plants, etc., and clean solid ground is laid bare around the separate larger springs and around groups of smaller springs. The springs are then surrounded with concrete or brick walls higher than the ground, and a cover is provided to prevent impurities from getting in. In most cases artificial drainage requires pumping, but with springs issuing from sand, pumps must not be operated with such force that the springs begin to yield much sand, since otherwise underground conditions may be changed in time and the outlet of the spring may be shifted. A larger area of springs, with numerous little springs, is enclosed by a wall, after the pool has been cleaned. This wall is built above the level of the surrounding ground. On the solid ground surrounding the spring area, the individual springs are connected with one another by means of little channels. If the spring area is not too extensive, it is covered with an arch roof. Very extended spring areas are enclosed only by a masonry wall, or in loose ground, by sheet piles. Channels from the individual springs communicate with a collecting reservoir, from which the

water is taken when needed and which is covered and closed. Ventilation, as well as protection against frost and heat, must be taken into consideration in this case, as in all problems of spring construction, and means for emptying and overflowing should be provided.

Springs of Baden-Baden.[2a] Along the open cleavages between the granite and colored sandstone, there appear everywhere springs or threads of water. These are either caught separately or, wherever they appear in rows, are collected in conduits. Collecting conduits with a total length of 0.75 mi., are connected together by intermediate conduits of 0.5 mi. length. The conduits are 2 ft. 4 in. wide and 5 ft. 3 in. deep. Where a stronger spring issues, a

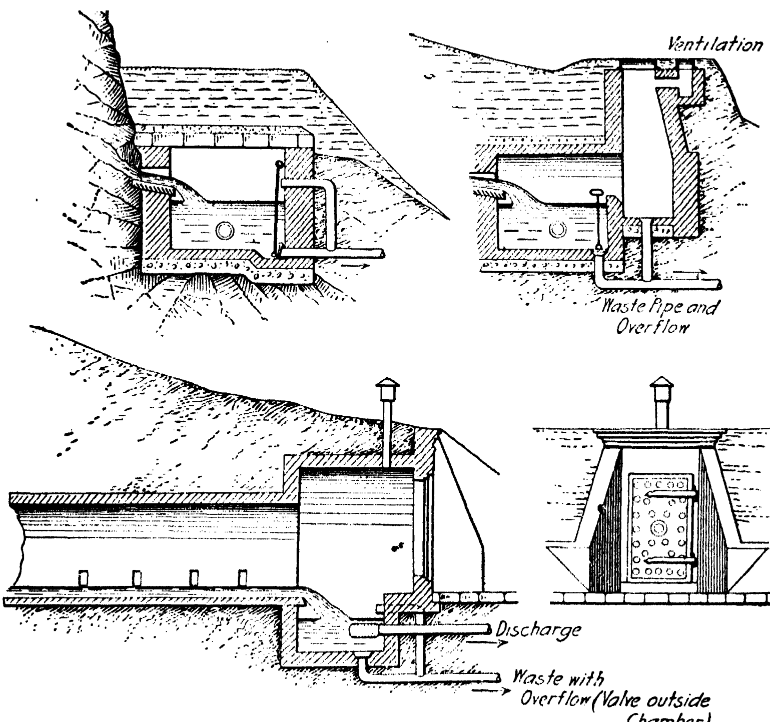


FIG. 114.—Spring houses.[3b]

niche is broken through the wall of the conduit; an opening is also made in the collecting pipe below the bottom of the conduit. Smaller threads of water are collected in clay pipes along the walls of the conduits, connecting with the next niche where water flows into the main collecting conduit. The bottom of the conduit serves only to collect the water dripping down from the walls; it runs from here to the spring house and then into the exit provided. The walls are built dry; the bottom, and side walls are covered with cement to a height of 8 in.

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PART III

TRANSPORTATION AND DELIVERY OF WATER

CHAPTER 13

OPEN CHANNELS

Economy of open channels in contrast to conduits lies in low cost of materials utilized, saving of materials in the roof of the conduit, and elimination of refilling over conduit. Against these must be balanced evaporation and seepage losses,* the longer line required to locate the channel topographically to follow the hydraulic grade line, ice troubles in regions of cold winters, and facility of pollution. Cattle and tree roots have injured concrete linings;[24] gopher holes weaken the foundation. Cross-drainage must be cared for.

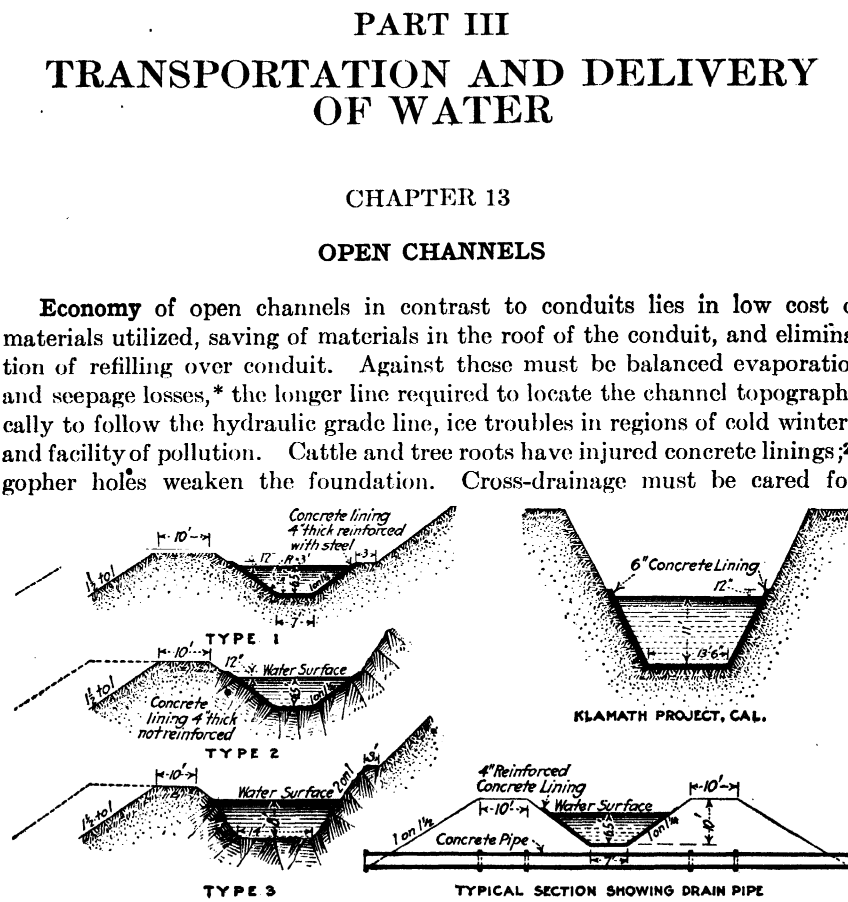

FIG. 115.—Open channels. U. S. Reclamation service.

Forms of Section. For semicircular section running full, or for lower half of any regular polygon, also running half-full, hydraulic radius equals half radius of inscribed circle. Any such half regular polygon has minimum wetted perimeter for given area, and, consequently, is of most advantageous form from a theoretical point of view to deliver maximum quantity of water for given slope of bed, given area of water prism, and given number of sides for polygon. Of all trapezoidal sections running full and having a common side slope, or angle θ from the horizontal, that one is of most advantageous form whose three sides forming the wetted perimeter are tangent to the semicircle having a radius equal to the depth and with its center in the surface of the water; its

* Assumed at 6 cfs. per million sq. ft. of wetted area on unlined canals for Lethbridge Northern Irrigation District.[26]

hydraulic radius is equal to half the depth.[1] The half-hexagon is the most economical trapezoid.[11]

The side slopes given to unlined ditches should be based on successful practice and should recognize climatic influences, particularly frost and ice action. The type of excavating machinery also determines the slopes. Etcheverry[2] cites as common practice:

For cuts in firm rock	1 on $\frac{1}{4}$
For cuts in fissured rock, disintegrated rock, tough hardpan	1 on $\frac{1}{2}$
For cuts in cemented gravel, stiff clay soils, ordinary hardpan	1 on $\frac{3}{4}$
For cuts in firm, gravelly, clay soil, or for side-hill cross-section in average loam	1 on 1
For cuts or fills in average loam or gravelly loam	1 on $1\frac{1}{2}$
For cuts or fills in loose, sandy loam	1 on 2
For cuts or fills in very sandy soil	1 on 3

On Lethbridge North Irrigation District,[26] canal sections were proportioned for $b = d^2 - dh$, in which b = base width, d = depth of water, and h = side slope ratio.

Table 70. Sections and Slopes of Some of the Principal Canals in Earth of the U. S. Reclamation Service[4]*

Project	Name of canal	Size, ft. Bottom width	Size, ft. Depth	Side slopes	Velocity	Capacity, cfs.	Slope	Kutter's n
Truckee-Carson	Main Truckee	20	13	$1\frac{1}{2}$:1	2.96	1,520	0.000154	0.025
Truckee-Carson	Main Truckee	12	13	1 :1	3.70	1,202	0.0003	0.025
Truckee-Carson	Lateral Line AA	13	6	2 :1	2.53	380	0.0002	0.020
Truckee-Carson	Lateral Line F	4	3	2 :1	1.65	50	0.00025	0.02
Truckee-Carson	Lateral Line I	6	3	2 :1	1.73	62	0.0002	0.02
No. Platte	Interstate	34	10	$1\frac{1}{2}$:1	2.86	1,407	0.00017	0.025
No. Platte	Interstate	24.5	10	2 :1	2.94	1,220	0.00017	0.025
No. Platte	Interstate 2nd	30	9.5	$\frac{1}{2}$:1	2.75	908	0.00017	0.025
No. Platte	Interstate 2nd	22	8.5	$1\frac{1}{2}$:1	2.50	738	0.00017	0.025
Lower Yellowstone	Main	23.5	10	$1\frac{1}{2}$:1	2.14	824	0.0001	0.025
Lower Yellowstone	Main	23	8	$1\frac{1}{2}$:1	1.89	529	0.0001	0.025
Lower Yellowstone	Main	15.5	6	$1\frac{1}{2}$:1	2.12	312	0.002	0.025
Huntley	Main	14.5	7	$1\frac{1}{2}$:1	2.29	401	0.0002	0.025
Huntley	Main	11	5	$1\frac{1}{2}$:1	2.60	240	0.0004	0.025
Huntley	Main	8	3	$1\frac{1}{2}$:1	2.62	98	0.0008	0.025
Huntley	Main	5	3	$1\frac{1}{2}$:1	2.00	56	0.00055	0.025
Huntley	Main	4	2	$1\frac{1}{2}$:1	1.52	21	0.00055	0.025
Uncompahgre	South	30	10	2 :1	2.5	1,248	0.000135	0.025
Uncompahgre	South	40	8.3	2 :1	2.5	1,175	0.00015	0.025
Belle Fourche	North	40	9	1 :1	3.08	1,358	0.0002	0.025
Belle Fourche	North	27	7	$1\frac{1}{2}$:1	2.51	659	0.0002	0.025
Belle Fourche	North	23	7	$1\frac{1}{2}$:1	2.45	575	0.0002	0.025
Belle Fourche	South	20	5	1 :1	2.47	307	0.0003	0.025
Belle Fourche	South	16	5	$1\frac{1}{2}$:1	2.36	278	0.0003	0.025
Belle Fourche	Waste Channel	50	15	1 :1	3.00	2,926	0.0001	0.025
Belle Fourche	Inlet	40	10	1 :1	3.27	1,635	0.0002	0.025
Belle Fourche	Inlet	36.6	10	$1\frac{1}{2}$:1	3.22	1,659	0.0002	0.025
Klamath	Main	44	11	$1\frac{1}{2}$:1	2.25	1,500	0.000081	0.025
Klamath	E. Branch	16	6	$1\frac{1}{2}$:1	1.74	261	0.000132	0.025
Umatilla	Feed	13.0	5.5	2 :1	2.19	300	0.000201	0.0225
Umatilla	Feed	11.8	5.5	$1\frac{1}{2}$:1	2.18	300	0.000193	0.023
St. Mary	Main	27	8	$1\frac{1}{2}$:1	2.70	850	0.0002	0.025
Shoshone	Garland	40	6.5	2 :1	2.91	1,000	0.00022	0.0225
Shoshone	Garland	20	9.7	2 :1	2.58	1,000	0.00016	0.025

* See also Scobey, U. S. Dept. Agri., *Bull.* 194, 1915, p. 19–27.

Lining.* Where water is valuable, concrete lining is economical in saving both seepage and maintenance; where water is plentiful, the expense may not be justified. Metal flumes are sometimes incorporated as linings. With concrete lining, a slope of 1 on 1.5 may be used unless there is probability

* See also p. 538.

of the canal being emptied, with saturated material back of the lining. Where freezing may occur, the upper part of the lining should be made heavier and backed with free draining material. Slopes steeper than 1 on 1½, generally require over a 2-in. thickness. Thicker linings with reinforcement are required in unfavorable places—swamps, fills, etc. Linings should contain provisions for expansion. In mild climates, thin linings have been used successfully. Gage Canal, near Riverside, Cal., 20 mi. long, has a lining ¾ in. thick; in service 10 years.[2c] On the Umatilla project, 1½-in. thickness has given satisfactory results. Etchverry[2d] cites a 3-in. lining as eventually more economical than a 2-in. One- to two-in. linings have proved successful where frost action is not severe.[24] Oiling has been tried in California and India.

FLOW-LINE CALCULATIONS

Critical depth is that depth of flow at which the moving mass is in unstable equilibrium, and seeking either a higher or lower stage. It is that stage at which velocity head equals ½ depth, or, expressed otherwise, is that depth at which depth + $\frac{V^2}{2g}$ has the minimum value. Obviously it is independent of values of Kutter's n. The plotting of the critical depth on profiles is a helpful guide in choosing successive values of depths in cut-and-try calculations of flow. The flow line intersects the line of critical depth only at points where a break in grade, or other obstruction, causes a sudden change in velocity head. In passing from a low to a high stage, unless the waterway is properly proportioned, a hydraulic jump is liable to occur. Velocity at critical depth is termed critical velocity.*

Caution in Use of Formulas. For the practical application of any formula to an important new case, the engineer should always consult actual gagings, approximating as nearly as possible to the new case, whenever this can reasonably be done. Records of gagings are published in Ganguillet and Kutter (Hering and Trautwine's translation, Wiley, 1888); in Part IV. *Technical Reports*, Miami Conservancy District (I. E. Houk), 1918; and in "Hydraulic Flow Reviewed," by A. A. Barnes (Spon & Chamberlain, 1916).

Kutter's Formula for Uniform Motion.† The engineers of Miami Conservancy District, after study of all formulas for open-channel flow, concluded that Kutter's formula is the best at the present time; and that it is based on essentially correct principles.[5] It may be used for channels, pipes, and conduits.

$$V = \left[\frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.6 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}$$

R = hydraulic radius. S = slope. V = velocity. n = effect of roughness of channel.

n = 0.009, for well-planed timber evenly laid.
0.010, plaster in pure cement; glazed surfaces in good order.
0.011, plaster in cement with one-third sand; iron and cement pipes in good order and well laid.

* Horton[20] distinguishes between this meaning and that of Reynold's (where flow changes from linear to turbulent), or Kennedy's (Proc. Inst. C. E., Vol. 119, 1895, p. 287), where silting or scouring action is impending.

† For eroding velocities in unlined channels, see p. 106.

0.012, unplaned timber, evenly laid and continuous.
0.013, ashlar masonry and well-laid brickwork; also the above when not in good condition nor well laid.
0.015, "canvas lining on frames;" brickwork of rough surface; foul iron pipes; badly jointed cement pipes.
0.017, rubble in plaster or cement in good order; inferior brickwork; tuberculated iron pipes; very fine and rammed gravel.
0.020, canals in very firm gravel; rubble in inferior condition; earth of even surface.
0.025, canals and rivers in perfect order and regimen and perfectly free from stones and weeds.
0.030, canals and rivers in earth in moderately good order and regimen, having stones and weeds occasionally.
0.035, canals and rivers in bad order and regimen, overgrown with vegetation, and strewn with stones and detritus.
0.060 to 0.10, not uncommon for river channels.[6]

For other values, see Dept. of Agri. *Bull.* 194 (1915), 376 (1920) and 852 (1920) by F. C. Scobey; *Bull.* 832 (1920) by Ramser; and Catalog of R. Hardesty Manufacturing Co., Denver.

Algae growths increased n from 0.012 to 0.014 on the concrete-lined Tieton canal.[7] Insect larvae clinging to sides, with velocities as high as 9 ft. per sec., have decreased capacities.[8]

Rock Channels. In 1908, J. C. Stevens[9] made rating of Umatilla River, Ore., where flow is over water-worn basalt rock, free from silt, sharp corners, and jagged edges. Values in Table 71 were derived from rating curve. Stevens suggests no less a value than n = 0.035 for unlined canals in newly excavated rock.

Table 71. Gaging of Flow in Water-worn Rock Channel

Gage height, ft.	Area, sq. ft.	Mean vel., ft. per sec.	Discharge, cfs.	Slope	Hyd. radius, R ft.,	Chezy, C	Kutter, n
4	325	4.24	1,380	0.00435	1.87	47.0	0.0340
5	505	5.86	2,960	0.00415	2.77	54.7	0.0323
6	688	7.14	4,910	0.00395	3.70	59.0	0.0316
7	875	8.35	7,300	0.00375	4.58	63.8	0.0305
8	1,065	9.58	10,200	0.00355	5.46	68.8	0.0294
9	1,255	10.43	13,100	0.00335	6.32	71.7	0.0285

Ramser[25] found values of n = 0.0385 to 0.0420 for rough rock channels excavated by explosives.

Chezy Formula applies to both pipes and open channels. $V = C\sqrt{R}\sqrt{S}$. V = velocity, ft. per sec.; R = hydraulic radius, ft.; S = slope = sine of angle water surface makes with the horizon. C has the value of the bracketed factor in Kutter's formula, p. 271. Values of C for various values of Kutter's n, R, and S are given in Trautwine's "Engineers' Pocketbook," and in King's "Handbook of Hydraulics." Kutter's expression for C is empirical and should be used with caution, keeping within limits based upon reliable data. On the circular concrete-lined Catskill aqueduct with R = 3.5 and S = 0.022, tests indicate values of C from 135 to 155. A value of 125 was basis of design.

Calculations for steady, non-uniform flow,*† *i.e.* constant quantity through a channel having cross-sections of various areas and shapes. In any reach of constant cross-section, water-depth will be constant, but it will differ in reaches having different cross-sections. Introducing 10,000 into the numerator and denominator of Merriman's formula[10] to obviate awkward decimals, and calling $\sqrt{2g} = 8$, we obtain:

$$Q = 80{,}000 \sqrt{\frac{h}{\left(\frac{10{,}000}{A_U^2} - \frac{10{,}000}{A_D^2} + \frac{644{,}000 \times L \times P}{C^2 A^3}\right) 10{,}000}}$$

in which

h = drop in ft., of water surface, in distance L, ft.
D_D = average depth at downstream section.
D_U = average depth at upstream section.
A_U = area at upstream section, sq. ft. = $D_U \times W_u$
W_U = average width at upstream section.
W_D = average width at downstream section.
A_D = area at downstream section sq. ft. = $D_D \times W_D$
C = Chezy's C, average value.
P = wetted perimeter at average area, ft.
A = average area, sq. ft.
Q = quantity, cfs.

The quadratic form of this equation‡ indicates that there are two values of depths (or areas) which may satisfy it. To choose the proper depths in the initial downstream section for a channel where no sharp break in bottom slope occurs, plot a curve with A_D constant at, say, A'_D, and, for various assumed values of A_U, compute values of Q. Plot these to abscissa scale of Q and ordinate scale of D_U; obviously this curve will be either tangent or asymptotic to ordinate representing level water surface, ($h = O$). There will be one point only on this curve, the vertex, where but one value of the depth satisfies the equation. Likewise take a second value of A_D, and for assumed values of A_U plot a second curve of Q. (Caution: If A_U is not taken sufficiently large, Q becomes imaginary.) If these curves are properly chosen, the vertices will fall either side of the value of Q (say Q_1) for which channel is to be designed. A curve (an envelope curve for level invert), drawn through these vertices, is the locus of the single values of D_U that satisfy the equation; the point where the envelope curve intersects the specified value for Q indicates the proper values for D_U and D_D; D_D is read by interpolation. Proceed similarly for next section upstream, using previous values of D_U and A_U as D_D and A_D, respectively. Assume values of D_U, until equation is satisfied that Q has the specified value.

* Backwater calculations are the reverse of these.
† See also analysis by Babbitt, *E. N. R.*, Dec. 21, 1922, p. 1067; Husted, *E. N. R.*, Apr. 24, 1924, p. 719; and Hill, *E. N. R.*, Apr. 19, 1923, p. 707; the last is applicable to drop-down curves in sewers.
‡ Note that this equation takes account of the difference of velocity heads by means of the first two terms in the denominator; and that the third term (always positive) represents friction, and that it may be enlarged to embrace all other losses.

Allowances for other losses of head, due to changes in section, admission at sides of streams with different angles and velocities of entrance, changes in quantities, etc., should be made as follows:

Change in section = between $\frac{(V_1 - V_2)^2}{2g}$ and zero, according to design.

Entrance of stream at side = between $\frac{V^2}{2g}$ and zero, according to design.

Change in quantity $= \frac{V_1^2}{2g} - \frac{V_2^2}{2g}$.

Starting loss $= \frac{V^2}{2g}$.

Loss due to entrance at upper end of canal $= 0$ to $\frac{1}{2}\frac{V^2}{2g}$.

Hydraulic Jump.* (Formula deduced by Unwin; also by S. M. Woodward in *Miami Technical Reports*, Part III, 1918.)

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}}$$

Where
D_1 = initial depth of stream, ft.
D_2 = final depth of stream, ft.
V_1 = initial velocity, ft. per sec.

Extremely high jumps tend to be unstable in position and action. A method of locating is given by Julian Hind, *E. N. R.*, Nov. 25, 1920, p. 1034. Hydraulic jump may be expected wherever an obstruction to flow, such as a break in grade, exists, if V_1 exceeds $\sqrt{gD_1}$; it occurs only when D_1 is less than the critical depth. "If investigation of a proposed channel indicates the depth may be at or near the critical, the shape or slope of the channel should, if practicable, be changed to secure greater stability.' Usually the critical velocity can be changed by widening or narrowing the channel, or the normal velocity by altering the slope. If such changes are not practicable, liberal freeboard should be allowed, and extreme care should be used in construction to secure uniformity in grade and cross-section."[27]

Absorption of High Velocities. Ewald[21] proportioned stilling pools for Cheoah dam and Narrows dam on Yadkin River as follows:

Minimum depth $= 2\sqrt{\frac{D_u V_u^2 . 2}{2g}}$

Minimum depth is measured above same datum as D_u. D_u is depth of flow in channel at entrance to pool; V_u is velocity at the same point.

The minimum length to absorb bottom velocity, $V_u = 1.2V_u^2/2g \times 1.35$. Width of pool need not exceed width of channel, but should not be less.

Mean Depth or Mean Radius.[12] The mean depth of water is expressed usually as "mean depth" for rivers, and as "mean radius" for artificial channels. There is a difference, sometimes of moment, between these two expressions. Applications of formulas, therefore, should take due account of it.

* See also "Finding Critical Depth", by Bailey, E. N. R., Nov. 12, 1925, p. 810; "Determining Energy Lost in Hydraulic Jump", *E. N. R.*, June 4, 1925, p. 928.

Comparison of Mean Depth and Mean Radius

$$\text{Mean depth} = \frac{\text{Area}}{\text{Width}} = \frac{12}{4} = 3.0$$

$$\text{Mean radius} = \frac{\text{Area}}{\text{Wet perim}} = \frac{12}{10} = 1.2$$

$$\text{Mean depth} = \frac{\text{Area}}{\text{Width}} = \frac{12}{6} = 2.0$$

$$\text{Mean radius} = \frac{\text{Area}}{\text{Wet perim}} = \frac{12}{10} = 1.2$$

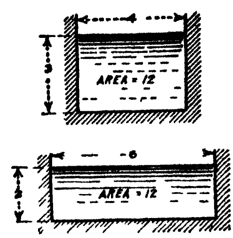

FIG. 116

Table 72

Hydraulic elements	Sections				
	C	D	E	F	All
Length, ft	640	120	220	1075	2055
Slope	0 0014	0 0017	0 0027	0 0017	0 0018
Average depth of water, ft	4 00	4 10	4 02	3 96	3 97
Mean area, sq ft	28 95	29 90	29 10	28 50	28 64
Mean hydraulic radius, ft	2 13	2 18	2 17	2 11	2 13
Quantity, cu ft per sec	----	----	----	----	205
Velocity, ft. per sec	7 1	6 8	6 9	7 1	7 2
Chezy's C (page 265)	129	114	90	119	117
Kutter's n	0 0132	0 0149	0 0189	0 0142	0 0146

Gagings in a Concrete-lined Channel, Umatilla Project, U. S. Reclamation Service. Results in Table 72. The channel section is semicircular, 9 8 ft diam ; ultimate capacity, 300 cu ft per sec The four sections tested make up the main canal, entry and exit curves being omitted Sections D and E are curves of 50-ft and 100-ft radii, which caused great disturbance of flow. Sharp curvature cannot be introduced into canal alinement at such frequent intervals as will lead to the overlapping of the backing-up effect, thereby inducing an actual net loss of head [13]

Surging in Open Channels. Surges in Tieton canal (8 ft. 3 in. diam., semicircular, concrete lined), seemed to obey no definite hydraulic law. With a maximum flow of 274 cu ft per sec , on some 75-ft stretches, water would be 3 ft deep, on other similar tangents, 5 ft deep. E G Hopson thinks that the canal was constructed with too rigorous a contraction at the headworks; at certain points variations in depth and velocity could not be explained by alinement or quality of the work Points of turmoil occurred at about 200-ft. intervals, regardless of curve or tangents, being somewhat more noticeable on tangents. In addition, there were traveling surges, 6 ft higher than the surrounding water, which worked slowly downstream, beginning at a point where the depth was 4 ft , with quantity of 200 cu ft per sec. On curves, the water often heaped on the inside edge. The velocity was high enough to start a 25-lb. stone downstream. A velocity of 11 ft per sec. caused waves of considerable size on uniform slopes. J. S. Conway believes that points of turmoil are generally caused by small irregularities in alinement and grade and traveling surges by loss and recovery of hydraulic equilibrium. H. F. Dunham believes that irregularities in the water surface may be to some extent explained by Spencer's Law of Rhythm.[14]

Curvature Effect. Centrifugal action causes the piling up of water at the concave shore. An approximation* of difference of level at the high side and at the center $= \frac{V^2b}{2gR}$, in which $\frac{V^2}{2g}$ = average velocity head; b = channel width; and R = radius of curvature of center line. Eddy concludes[22] that where Kutter's $n = 0.025$ for a straight channel, values of 0.028 to 0.030 should be used where the curvature is considerable.

Table 73. Maximum Mean Velocities Safe against Erosion[2b]

Material	Mean velocity in feet per second
Very light pure sand of quicksand character	0.75– 1.00
Very light loose sand	1.00– 1.50
Coarse sand or light sandy soil	1.50– 2.00
Average sandy soil	2.00– 2.50
Sandy loam	2.50– 2.75
Average loam, alluvial soil, volcanic ash soil	2.75– 3.00
Firm loam, clay loam	3.00– 3.75
Stiff clay soil, ordinary gravel soil	4.00– 5.00
Coarse gravel, cobbles, shingles	5.00– 6.00
Conglomerates, cemented gravel, soft slate, tough hardpan, soft sedimentary rock	6.00– 8.00
Hard rock	10.00–15.00
Concrete	15.00–20.00†

† In spillway channels, 50 ft. is not uncommon.

Inlets and outlets connecting canals to flumes or "siphons" should be so proportioned that the loss or restoration of velocity head is accomplished with minimum disturbance. Books on irrigation should be consulted for these and other structures, improved in irrigation practice.

Regulating Dams. Dams in the Wachusett open channel (Metropolitan W. W., Boston) for regulating velocity and providing for drawing water from beneath ice have been satisfactory. With low velocities (less than 4 ft. per sec.) gravel affords sufficient protection for the banks, but shoving of ice mars the water lines so that for appearance they need to be restored each spring. Depths of water flowing over such dams should be kept low; this frequently necessitates length of crest greater than width of channel; by building the dam in form of arc or V, extending upstream or downstream, any desired crest length can be obtained. Sluiceways controlled by gates, below ice level, make conveniently possible the drawing of water without disturbing the ice sheet or causing ice jams.

FLUMES

Flumes were evolved to carry open channels across depressions, but have since been employed in other topography to eliminate earthwork, to afford closer control of leakage, and to provide a higher head for irrigation distribution systems.

Wooden flumes vary widely in size and in condition of service, consequently in details of design. Yellow pine is one of the best materials. Where single thickness only is used on sides, joints should be closed by battens

* Formula derived by Prof. S. M. Woodward, Part VII, *Technical Reports*, Miami Conservancy District, 1920.

or by splines. Wooden flume sections are given in books on irrigation, and in *E. N.*, July 22, 1915, p. 172; also Sept. 14, 1916, p. 540. A successful design is shown in Fig. 117. To maintain tightness of joints and prevent decay of wood, flumes should be kept full of water, or at least thoroughly wet. If necessarily empty or partially empty at times, wooden flumes should be sheltered from hot sun and drying winds by an inexpensive roof and side sheathing.

Semicircular flume of creosoted Douglas fir staves was built in 1923 for Central Oregon Irrigation Dist.[23] Bids also taken for metal flume. Diam. 12 ft. Velocity 11.6 ft. per sec. After milling, staves are creosoted under 8-lb. pressure.

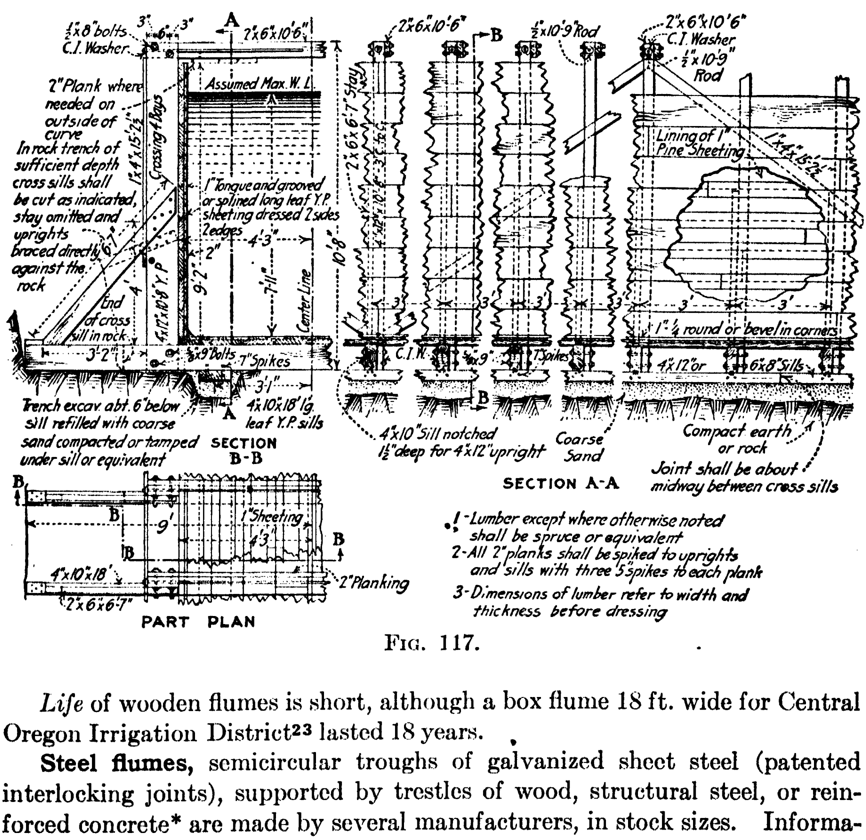

FIG. 117.

Life of wooden flumes is short, although a box flume 18 ft. wide for Central Oregon Irrigation District[23] lasted 18 years.

Steel flumes, semicircular troughs of galvanized sheet steel (patented interlocking joints), supported by trestles of wood, structural steel, or reinforced concrete* are made by several manufacturers, in stock sizes. Information on the Hess flume made by R. Hardesty Co., Denver, Colo. (Hess, Lennon, Maginnis, and Williams flumes), is given in Table 74 as more or less typical. From hydraulic viewpoints, preference should be given to a type of flume free from ridges or projections in the waterway.

Experience of U. S. Reclamation Service with metal flumes is epitomized by Julian Hind, in *E. N. R.*, May 25, 1922, p. 854. Their life may be placed at 10 to 25 years when properly protected. Requisites for durability learned from 69 flumes on Uncompahgre project are:[15] (1) proper proportioning

* Reinforced-concrete trestle is shown in *E. N. R.*, May 15, 1919, p. 978.

of details; (2) metal thick enough to resist deformation; (3) thorough galvanizing;* (4) coating maintained; (5) sand and gravel trapped out; (6) provision for temperature movement; (7) self-drainage.

Reinforced concrete flumes may have rectangular cross-section wherein the side walls are designed either as cantilevers as on San Joaquin flume, Cal.[16] or as supported top and bottom as on King Hill project, Idaho,[17] or may have a section conforming to the hydrostatic catenary, as at Brooks aqueduct.[18] The advantage of the last lies in the fact that there is neither moment nor shear at any point.[18] Gunite has been used, notably on King Hill project.[17]

Table 74. Flume Data and Sizes of Timber Substructures for All Sizes of Improved Hardesty Metallic Flumes
(For height of 12 ft. and 16 ft. center to center bents)

Flume No.	Diameter, ft., in.		Area, square feet	Total weight per foot complete (flume full of water), pounds	Size of substructure timbers, inches: Posts	X braces	Knee braces	Stringers†	Dist. betw'n stringers, ft., in.		Carrier beams*: Size, in.‡	Total length, ft. in.		Dist. c.-c. of holes for carriers, ft. in.		Ft. B.M. per linear foot of flume: Sub-structure	Carrier b'ms	Total
12		7⅝	0.16	15	2× 4	1× 4	1× 4	2× 4		9½	a	1	2		8½	3.44	.04	3.48
15		9½	0.24	22	2× 4	1× 4	1× 4	2× 4		11½	a	1	4		10½	3.44	.04	3.48
18		11⅜	0.35	41	2× 4	1× 4	1× 4	2× 4	1	1½	b	1	6	1	0½	3.44	.10	3.54
24	1	3⅛	0.62	55	4× 4	2× 4	2× 4	2× 6	1	5	b	1	10	1	3¼	7.1	.12	7.22
30	1	7⅛	0.99	80	4× 4	2× 4	2× 4	2× 6	1	10	c	2	4	1	8½	7.1	.23	7.33
36	1	10⅞	1.43	109	4× 4	2× 4	2× 4	2× 6	2	1	c	2	8	2	0½	7.1	.27	7.37
42	2	2¾	1.95	135	4× 4	2× 6	2× 6	2× 6	2	5	d	3	0	2	3½	8.27	.39	8.66
48	2	6½	2.53	171	4× 4	2× 6	2× 6	2× 6	2	10½	d	3	4	2	7	8.27	.44	8.71
60	3	2¼	3.97	247	4× 4	2× 6	2× 6	2× 6	3	6¼	d	4	0	3	3	8.35	.88	9.23
72	3	9¾	5.72	387	6× 6	2× 6	3× 6	3× 8	4	1¾	d	4	8	3	10½	14.2	1.03	15.23
84	4	5½	7.81	533	6× 6	2× 6	3× 6	3× 8	4	9¾	e	5	6	4	6	14.3	1.83	16.13
96	5	1⅛	10.17	683	6× 6	2× 6	3× 6	3× 8	5	5⅛	e	6	0	5	1¾	14.4	2.00	16.40
108	5	8¾	12.84	860	6× 6	2× 6	3× 6	3× 8	6	1	e	6	10	5	9½	14.5	2.28	16.78
120	6	4⅜	15.90	1088	6× 6	2× 8	3× 8	3×10	6	9⅜	f	7	6	6	5½	17.2	3.33	20.53
132	7	0	19.24	1303	6× 6	2× 8	3× 8	3×10	7	4¾	f	8	0	7	0¼	17.3	3.54	20.84
144	7	7¾	22.92	1517	6× 6	2× 8	4× 8	3×12	8	0½	f	9	0	7	8⅛	19.9	3.99	23.89
156	8	3¼	26.87	1783	8× 8	3× 8	4× 8	3×12	8	9⅜	f	9	6	8	5	27.3	4.21	31.51
168	8	10⅞	31.12	2065	8× 8	3× 8	4× 8	3×12	9	4⅜	f	10	0	9	0	27.5	4.43	31.93
180	9	6⅝	35.80	2369	8× 8	3× 8	4× 8	3×12	10	0¾	f	11	0	9	8	27.7	4.88	32.58
192	10	2¼	40.69	2675	8× 8	3× 8	4× 8	6×10	10	8	f	12	0	10	3¼	29.9	5.32	35.22
204	10	9⅞	45.97	3012	10×10	3×10	4×10	6×10	11	4⅛	f	13	0	10	11⅛	37.7	5.76	43.46
216	11	5½	51.27	3425	10×10	3×10	4×10	6×12	12	0⅝	g	14	0	11	7¼	40.2	9.33	49.53
228	12	1¼	57.49	3745	10×10	3×12	4×12	6×12	12	7¾	f	15	0	12	2⅜	42.9	13.30	56.20
240	12	8¾	63.68	4145	10×10	3×12	4×12	8×12	13	4	f	16	0	12	10½	48.0	14.20	62.20
252	13	4⅜	70.18	4629	12×12	3×12	4×12	8×12	14	0	f	16	6	13	6½	51.0	14.60	65.60

* For exceptional spans, carrier beams are trussed; see 32-ft. span in *E. N.*, July 20, 1916, p. 127.
† Stringers run between bents, outside of flume, and support transverse beams (carriers) from which flume is suspended.
‡ *a*, 1 × 2; *b*, 2 × 2; *c*, 2 × 3; *d*, 2 × 4; *e*, 3 × 4; *f*, 4 × 4; *g*, 4 × 6.

Contraction joints are not always installed. San Joaquin Light & Power Corp.[16] used none on flume 2½ mi. long, 6 ft. wide, having walls 3 ft. 6 in. high; but reinforcing was proportioned so that nothing but hair cracks would appear. "No visible cracks to date" (1923). Jointless flumes have a lower maintenance charge. Contraction joints were used on the reinforced concrete flume of the Northern Idaho and Montana Power Co., Big Fork, Mont. (10 by 22 ft.

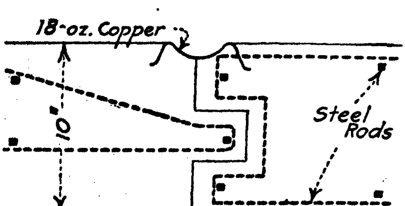

FIG. 118.

* 1½ oz. zinc coating was required on Lethbridge project.[26]

in cross-section), open to the air. It was necessary to design the contraction joints (Fig. 118) watertight as the freezing of water in the joints under a winter minimum of $-30°F.$ might cause spalling. A flexible copper covering was provided to permit movement as well as keep water out of the joint. An ordinary tongue-and-groove joint was used, reinforced to prevent breaking. The arc of copper was of sufficient length, so that at the lowest temperature, it would not be straight. The side of the copper next to the concrete was greased to prevent adhesion. The effectiveness is dependent on the adhesion of the ends of the copper to the concrete. Tests with 18-oz. copper inside a standard briquette, 1:2 mix, 28 days old, gave the following results:

Type	Pull to break concrete	Pull to break bond between copper and concrete
Without copper	240 lb.	....
Copper, cleaned	308 lb.	364 lb.
Copper, uncleaned	346 lb.	431 lb.

Approximately 2 sq. in. of copper was in contact; adhesion per sq. in. was 215 lb. with uncleaned copper and 182 with cleaned.[19]

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CHAPTER 14

AQUEDUCTS

MASONRY AQUEDUCTS

Terminology. The term "aqueduct," duct for water, is applicable to any form of closed waterway, steel, wood, reinforced-concrete, cast-iron or masonry conduits built in place, but it is commonly restricted to the last. Types are: (*a*) cut-and-cover, in open cut at hydraulic gradient; (*b*) grade tunnels, in tunnel at hydraulic grade; (*c*) pressure conduits, in open cut with top below hydraulic gradient; (*d*) pressure tunnels, below hydraulic gradient. Pressure conduits and tunnels are also designated "siphons" (because of shape of profile; they have no siphonic action). The types requiring reinforcing are tunnels in earth or poor rock, and pressure conduits; for the latter, metal pipes or reinforced concrete may be employed.

Use. Masonry aqueducts are commonly used in connection with large gravity water supplies. Cut-and-cover type is cheapest and should be employed wherever topography permits. There is economy in keeping as near as possible to hydraulic gradient. At valley crossings, pressure tunnels or pipe lines are substituted, and in crossing a divide recourse is had to the grade tunnel. Several lines of pipe of equivalent total capacity may cost much more than a pressure tunnel. Local conditions, especially geology, will determine. Pressure tunnels are "permanent;" pipes will deteriorate, although some kinds may have useful life of a century or more. Pressure tunnels have greater security in some respects, and are tighter. If full capacity is not needed immediately, there may be saving in first cost by adopting several lines of pipe, provided as needed.

Capacity required* depends on demand and on quantity of storage near point of distribution. When Winnipeg aqueduct was projected in 1913, the district had a population of 225,000. The aqueduct is proportioned for 102 mgd., equivalent to 120 gal. per day per capita to 850,000 people.[1] The nominal capacity of the Catskill aqueduct is 500 mgd., but it was designed to carry 603 mgd., to afford a factor of safety at times of excessive demand. As the life of an aqueduct is assumed much greater than a pipe line, it should be proportioned to meet demands in a more remote future.

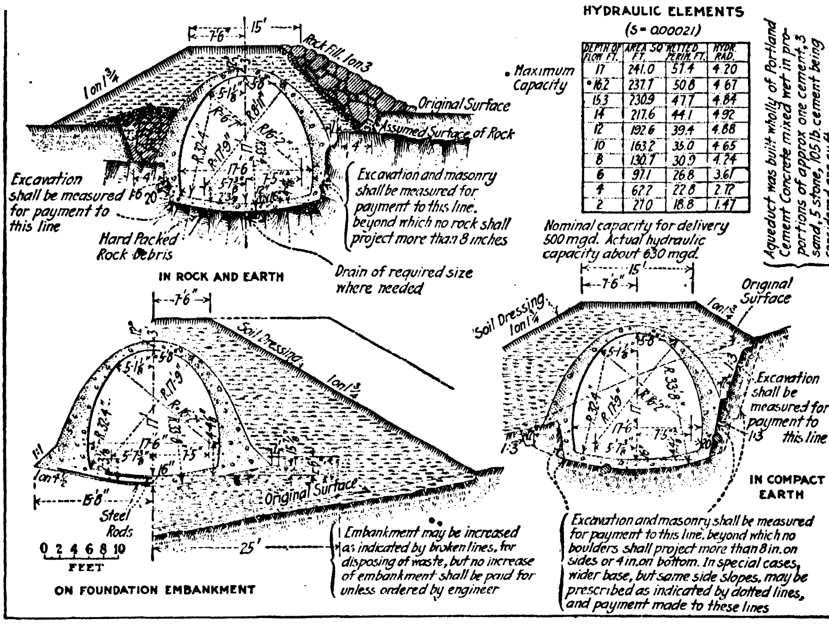
Profile. An economic study should be made to determine a size that will best fulfil the following conditions: (*a*) headworks arranged so that stored water may be drawn to lowest desirable level; (*b*) water delivered to city system at requisite pressure;† (*c*) pipe line gradient to be as steep as possible (see also p. 85). Condition (*c*) may be studied on a diagram for a given capacity by plotting slopes as ordinates and costs per foot as abscissas.

* Applies also to Chap. 15.
† At all rates of demand.

Costs should be balanced against value of storage sacrificed by raising outlet elevation at reservoir or against value of reduced pressure in distribution system.

Slopes Used on Catskill Aqueduct. Pressure aqueduct, 0.0023; cut-and-cover aqueduct, 0.00021; grade tunnel 0.00037; steel-pipe siphons, 0.00068; pressure tunnels, 0.0006115 to 0.00073. Wanaque aqueduct was designed on basis of 2 ft. per mi. Slopes on Winnipeg aqueduct[2] vary from 0.11 to 1.537 ft. per 1000, average 0.57.

Overflow weirs and spillways provide means for controlling the hydraulic gradient in flow-line conduits. If the quantity to be delivered exceeds the capacity of the section, the water backs up, filling the entire cross-section so that the capacity becomes even less, and section flows under pressure. Over-



Depth of flow ft.	Area sq. ft.	Wetted perim. ft.	Hydr. rad.
17	241.0	57.4	4.20
•16.2	237.7	50.8	4.67
15.3	230.9	47.7	4.84
14	217.6	44.1	4.92
12	192.6	39.4	4.88
10	163.2	35.0	4.65
8	130.7	30.9	4.24
6	97.1	26.8	3.61
4	62.7	22.8	2.72
2	27.0	18.8	1.47

FIG. 119.—Catskill aqueduct. Standard types, open cut and embankment.

flow weirs limit this action on Catskill aqueduct to 3 ft. head above the aqueduct arch.

CUT-AND-COVER AQUEDUCTS

Shape (see Figs. 119 to 123). Masonry aqueducts and similar conduits, unless reinforced with steel, are customarily built with approximately horseshoe cross-section, on account of good hydraulic properties, resistance to earth loads, facility and economy in construction, and convenience in maintenance. Aqueduct inverts should be dished only enough for convenience in cleaning and draining unless dishing is required to form an arch to resist side thrust at bottom of walls from earth load or to take upward water pressure.* In very large aqueducts it may be advisable to provide a level ledge on one side for a foot-way. In determining the thickness of inverts, allowance should be made for a few inches of concrete at the bottom, more or less mixed with dirt,

* Particularly when aqueduct is empty.

which will have but little strength, especially if placed directly on earth not very hard. The thicknesses of masonry in the Wachusett and Weston aqueducts are thought to be about the minimum desirable for aqueducts of such size, character, and conditions.*

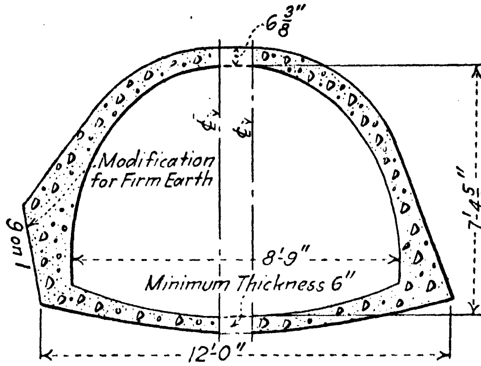


FIG. 120.—Winnipeg aqueduct.

Settlement of the Winnipeg aqueduct in clay soil which had lower supporting power than assumed, caused longitudinal cracks up to $\frac{1}{4}$ in. wide, in first season's work. Revised design widened the bearing area under the side walls in firm soil, and heavily reinforced the invert in compressible soils.[3] The cracks were repaired by cutting out to a depth of 1.5 or 2 in., a pear-shaped section, and calking with neat cement.

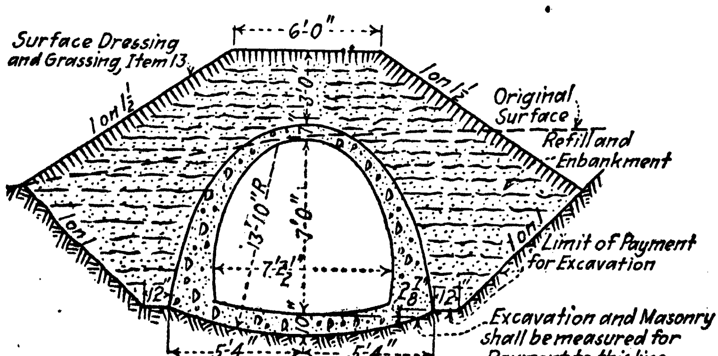


FIG. 121.—Wanaque aqueduct, type in dry loose earth.

Economics. A cut-and-cover aqueduct should be so located that the excavated material is just sufficient for the embankment requirements. Earth cover of 3 ft. above arch was required on Catskill aqueduct and 4 ft. on Winnipeg. The economic depth of cut will vary with the slopes in cut and with the width of top and slopes adopted for embankment.

* For Hartford aqueduct, see *E.R.*, Sept. 4, 1925, p. 290.

Masonry Aqueduct vs. Pipe Line. The economy in the aqueduct lies in its lower first cost and lower upkeep. It can be built of the materials of the region if suitable sand, gravel, and stone are found, and only the cement is dependent on commercial supply; whereas the price of cast-iron or steel pipe is determined by market conditions, freight rates, and other factors. The loss of carrying capacity with age is less in a masonry waterway than in one of

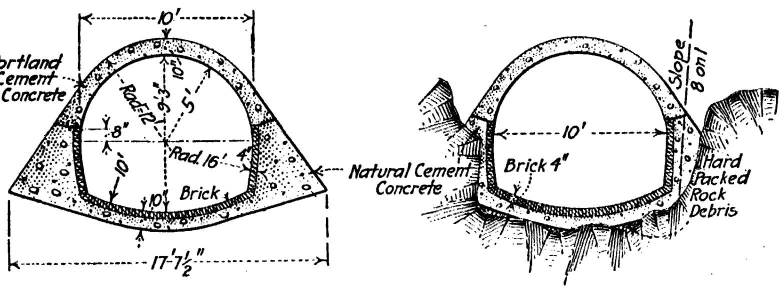


FIG. 122.—Weston aqueduct.
(Gradient 1 on 1250.)

metal. A longer life may be anticipated for the masonry. The balancing of cut and fill produces a structure above the natural surface, and requires culverts to take care of drainage and rights of way for highways and farm roads. Attractive plantings by abutting owners have met the complaint that the embankments are permanent eyesores. Aqueduct must be built full-size, whereas there is some economy in building a small pipe line first, and a supplementary pipe line when needed.

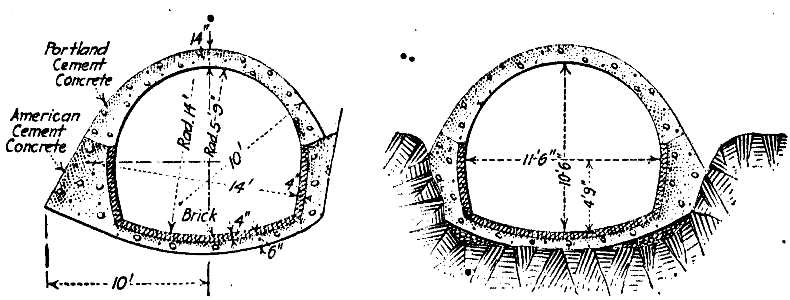


FIG. 123.—Wachusett (Nashua) aqueduct.

Materials. The older aqueducts were built of stone or brick, the recent of concrete.* For Catskill and Winnipeg mix was 1:2:4; later practice as on the Wanaque for Newark regulates the mix according to the graded aggregate. On the new conduit for Washington, a constant check on water content is kept, and a minimum time for mixing is maintained. Tests by Bureau of Standards indicate a strength of 2500 lb. per sq. in. at 28 days for the concrete

* *Construction methods* vary widely. See many articles in engineering press (1909–1912) on Catskill aqueduct. For Winnipeg aqueduct, see *E. N. R.*, Apr. 19, 26, and Mar. 8, 1917.

of the Washington conduit. On Winnipeg aqueduct, the sand and gravel were graded and mixed dry at a central point and delivered to mixers along the line, thereby saving 12 per cent. of cars.[6]

Alinement. Experience in construction and operation of the Wachusett and Weston aqueducts (Boston) shows no practical objection to sharp curves, radius depending somewhat on size of aqueduct; a radius of 100 to 200 ft. can be used on large aqueducts. Curves should be standardized and limited in number of radii, and forms should be well made, for convenience and economy. On Winnipeg aqueduct, curves were made up of 15-ft. tangents similar to steel pipe (see p. 331); 50-ft. radius was used.[7]

Changes in Cross-section. Experience indicates no serious objections to changes in cross-section of an aqueduct, if made by transition pieces without sudden enlargement or contraction. Such changes are sometimes desirable to conform to steeper or flatter slopes, to secure economical construction in tunnels, or for depressed portions under pressure, as beneath railroads or highways, and in similar places.

Culverts. Aqueducts and pipe lines having embankments above the natural ground surface intercept the drainage of the region, which must be conveyed in flumes over the aqueduct or in culverts beneath. Where free drainage is not feasible, siphon or dive culverts are employed, in which the outlet is lower than the inlet, and the middle is lower than either.* Means of access for cleaning must be provided. Culverts beneath large aqueducts should be of liberal size, none smaller than 4 sq. ft. area of waterway, preferably of concrete or other substantial masonry, and smooth inside. Experience indicates that culverts should not be fitted too closely to conditions existing when they are designed, since some culvert watersheds may later be increased in area by diversions, and others discharge more quickly because improved. In any region in which urban or suburban conditions may be brought about in even a remote future, culverts should be proportioned to care safely for very quick run-off. Place the invert of the culvert somewhat lower than the natural bed of the stream, to allow for future improvement of the stream which may lower the bed and increase the run-off. It is well to make culverts of such size and shape that they can be cleaned readily. For flows of ordinary occurrence, a velocity of 6 ft. per sec. should not be exceeded, in designing culvert openings, but occasional floods of 10 ft. per sec. are allowed so long as the channel leading away from the culvert is wide enough to reduce this velocity to the point of no erosion. With substantial paving between the wings of the culvert, and a short distance beyond, there would be no trouble from erosion. With velocities greater than 10 ft., the head at entrance causes an undesirable backing up of water. By selecting from list of coefficients following, values which suit the character of the country, an area can be determined safely and still not unreasonably large. Studies show that the Kuichling formula gives very large results compared with others for which coefficients suited to the Croton watershed are used. All formulas except Myer's give curves of the same general shape. The Talbot curve ($c = 0.3$) strikes a fair average of the U. S. Geological Survey, Burkli-Ziegler, and Fanning curves for $V = 8$, and other coefficients can be selected to average those

* See design in *E. N.*, Apr. 20, 1916, p. 755.

for other velocities. Culverts under Catskill aqueduct were proportioned according to Moore formula, velocities being limited to 60 ft. per sec. through the culvert; to date, areas of culverts have proved ample.

COMPARISON OF RUN-OFF* AND CULVERT FORMULAS

(a) *For Culvert Areas Direct.* Talbot: $A = c\sqrt[4]{D_a{}^3}$. Myer: $A = c\sqrt{D_a}$

(b) *For Run-off from Drainage Area.* Burkli-Ziegler. $Q_a = cR\sqrt[4]{\dfrac{S}{D_a}}$

Kuichling (Mohawk Valley): $Q_m = \dfrac{44,000}{D_m + 170} + 20$ (General)

$Q_m = \dfrac{127,000}{D_m + 370} + 7.4$ (Exceptionally heavy spring freshets)

United States Geological Survey (for Eastern U. S.).[8]

$$Q_m = \frac{46,790}{D_m + 320} + 15$$

Fanning (N. E. and Middle States):

$$Q = 200D_m{}^{\frac{5}{6}}.\quad Q_m = \frac{200D_m{}^{\frac{5}{6}}}{D_m}$$

$$\text{F. F. Moore}\quad Q_m = \frac{225}{D_m{}^{0.2}}.$$

A = Area of waterway, in sq. ft.
D_a = Drainage area in acres
D_m = Drainage area in sq. mi.
Q_a = Run-off in sec. ft. per acre
c = Coefficient, as follows
Q_m = Run-off in sec. ft. per sq. mi.
S = Average slope of ground, ft. per 1000.
Q = run-off in sec. ft.
R = Rainfall in in. per hr.

Myer. Hilly ground, 1.5. Mountainous and rocky ground, 4.

Talbot. Rolling agricultural country subject to floods at times of melting snow, and with length of valley three or four times width, $\frac{1}{3}$. Steep and rocky ground, $\frac{2}{3}$ to 1.

Burkli-Ziegler. (For run-off into sewers.) Paved streets, 0.75. Ordinary cases, 0.625. Suburbs with gardens, lawns, and macadamized streets, 0.31.[9]

Foundation Embankments. Embankments under Boston's aqueducts (Sudbury, 1878; Wachusett, 1898, Weston, 1903), have not settled perceptibly or so as to cause cracks. Slopes (1 on $1\frac{3}{4}$) of cover embankments have stood well. Weston aqueduct was built largely on embankment, thoroughly rolled in 4-in. layers of carefully selected materials. Banks stood 6 weeks or more before placing masonry. Some embankments are soaked by forming shallow pools on top when built to full height. After such soaking, the embankment should be given ample time to drain free of surplus water and harden before building masonry thereon. The thickness of layers may be 3 to 6 in., according to materials, height of embankment, etc. Each layer must be very solidly compacted. The embankment should be built 2 ft. or more above

* See also pp. 61 and 94.

final elevation, and the surplus excavated just before the masonry is built. The construction methods on Catskill aqueduct were similar. No perceptible settlements have occurred.

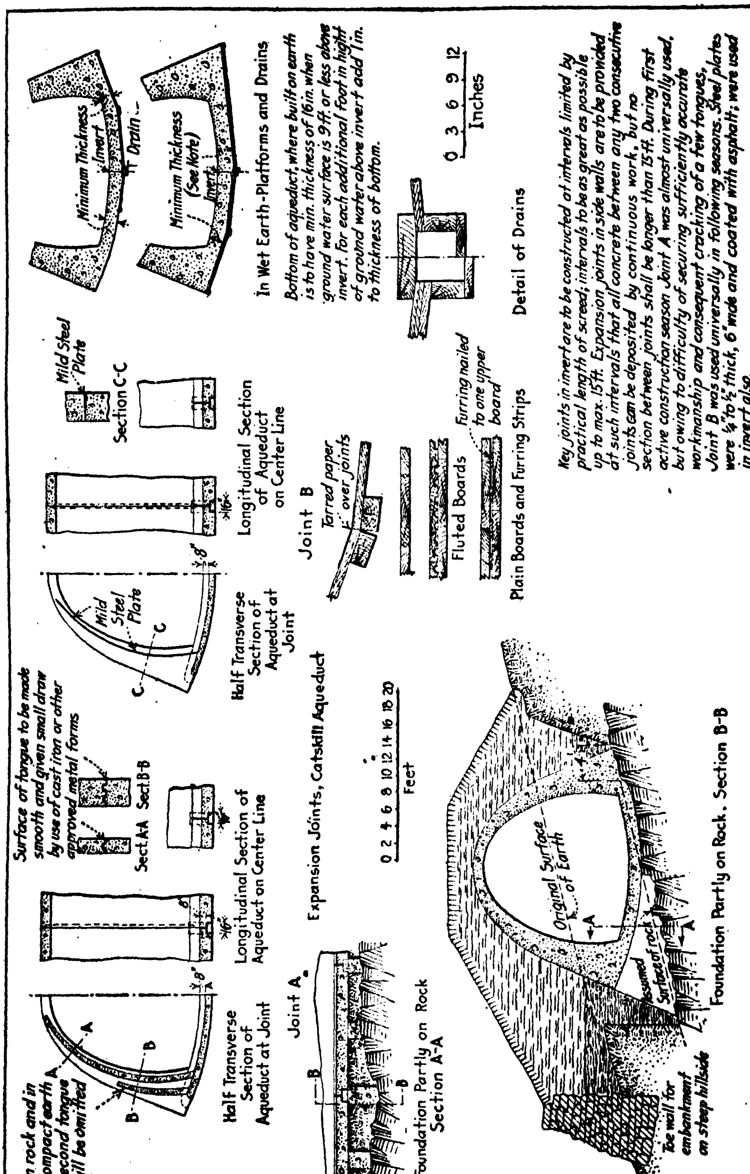


FIG. 124.—Catskill aqueduct. Standard details in open cut. Expansion joints and drains.

Cracks may be caused by overstressing due to temperature or settlement. Longitudinal cracks from settlement were troublesome on Winnipeg aqueduct (see *E. N.*, Oct. 12, 1916, p. 716); other aqueducts have suffered transverse cracks, of a minor character, due to temperature. In Catskill aqueduct,

transverse cracks have been observed in a few 75-ft sections but none in 60-ft sections. No cracks wider than 0.04 in. were found in the Wachusett aqueduct. In winter, the cracks were cut out by an expert mason with a diamond-pointed chisel, to a width of ¼ in., and a depth of 1¼ in. and pointed full with Portland cement mortar. Many parted again, but only so as to show an incipient crack at the surface. So far as observed, these secondary cracks do not go deeper than ¾ in., and no leakage took place.

Expansion (Contraction) Joints. Some engineers prefer to make no provision for expansion joints, but to treat such cracks as may occur. No expansion joints were provided in the Los Angeles aqueduct from Owens River. Copper sheets were used in Winnipeg aqueduct, at 45-ft. intervals.

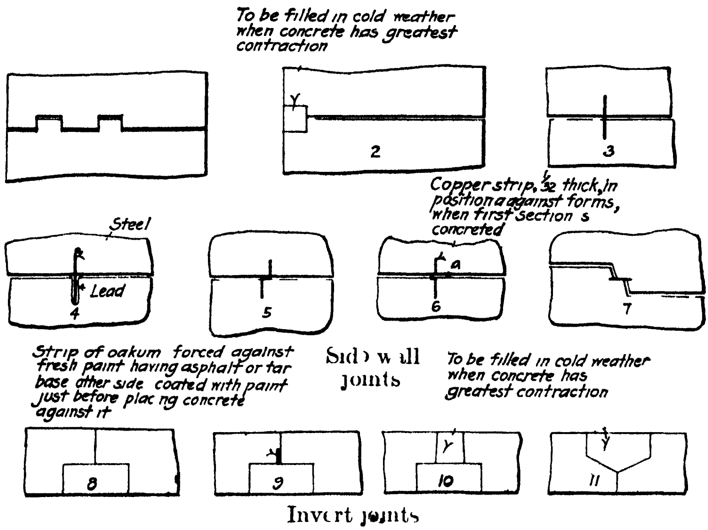


FIG. 125.—Expansion joints for cut-and-cover aqueducts.

The metal for joints 3 to 5 inclusive may be either mild steel or copper.* steel is suitable for joints 3 and 4 where metal is comparatively thick and copper is better for joints using thin metal on account of its non corrosive property and greater pliability. Lead which has been suggested for some of these joints does not offer sufficient resistance to damage and displacement while depositing concrete if of practicable thickness.

Joints 3 to 7 are applicable to the invert if used alone or in combination with a concrete key.

All contact surfaces between new and old concrete at expansion joints to be coated with paint having asphalt or tar base.

Expansion joints commonly require some such provisions as Fig. 125 to prevent objectionable leakage.

Hydrostatic tests on the newly completed portions of Catskill aqueduct showed that under severe conditions considerable leakage may occur at the joints. Tongue-and-groove expansion joints were used almost exclusively at first. As the groove forms had to be pulled in about 16 hr., due to the progress of the work, it was difficult to secure smooth sides and a true shape, which are essential if the tongue is to move freely in the groove. The small draw used in the tongue-and-groove joint, the relative narrowness of the tongue compared to depth, and the accidental skew of the bulkheads relative to the axis of the aqueduct, tend to cause a lock joint rather than a slip joint. Some tongues were found broken in cold weather. The consensus of opinion

* Ingot iron was used on Nepaug aqueduct, Hartford.

was that the tongue-and-groove joint was not practical, although giving excellent results when carefully made. Hydrostatic tests showed that leakage may be expected through invert joints. Consequently, for water stops in joints between days' work in arch and side walls, and between sections of invert, steel bands about $\frac{3}{8}$ in. thick by 6 in. wide, dipped or painted with asphalt, were adopted. Half the width was buried in the concrete first cast. Joints which opened were treated experimentally in many ways. The method adopted as simplest and most effective was to point the face of the crack with Portland cement mortar, and after this was hardened, to grout the space back of the pointing through a small pipe nipple, set near the bottom, using a small hand force pump (Douglas). In some cases, the joints were raked out. Grout was poured sometimes instead of being pumped.

Lead water stops did not prove wholly effective, and are not deemed worth the trouble and expense. An L-shaped joint smeared with asphalt, or some similar device, would probably be better than lead water stops.

Treating joints when open in cold weather gives the best results. Cement grout, lead wool, lead wire, hot tar, iron filings and sal ammoniac, oakum treated with pitch or asphaltum, or dipped in grout, all have been used. That leakage from masonry aqueducts cannot be wholly prevented is evident from universal experience. Although small quantities of silt, etc., carried in the water may reduce the leakage, even after many years some leakage still occurs, as can be seen by examining the older aqueducts. Careful construction can keep the leakage small. Expansion joints and cracks should have special treatment (pointing, calking, etc.) before the aqueduct is put in service, and subsequently, if necessary. Where the ground water is relatively high, the leakage may be inward.

The maximum opening for expansion joints will be determined by the range in temperature between the maximum reached during the placing of the concrete (95° ± F.) and the minimum subsequently reached (36° or lower). Weekly readings of water at the inlet and at the 135th St. gatehouse of the New Croton aqueduct, 1913 and 1914, showed a maximum of 74° and a minimum of 33°. Temperatures in 1923, of water drawn from an average depth of 75 ft., ranged from monthly average of 67° to 38°. Surface temperatures in Wachusett reservoir in 1923 ranged from 71° to 33°, while at 107 ft. depth, range was 60 to 35°. The extreme minimum in Catskill aqueduct is estimated at 36°, and the average minimum, 40°. The water temperature may reach 70° in summer, and will probably average 65°. Of course, local climatic conditions, size of conduit and depth of cover will affect these temperatures. For temperatures in Winnipeg aqueduct, see *J. A. W. W. A.*, Vol. 7, 1920, p. 702.

Temperature of water tends to change on Apulian aqueduct[10] at exposed crossings. To lessen this, an air space was inserted between aqueduct wall and parapet, and top of aqueduct covered with earth.

Forms. On Catskill aqueduct collapsible steel forms of Blaw, Ransome, and other makes were employed. The tendency at first was to make the forms too light, with poorly fitting joints (see Fig. 119, for section of this aqueduct). The greatest horizontal movement in the inside forms occurred about 10 ft. above the invert, and in some cases amounted to 0.02 to 0.14 ft.,

depending on design. The maximum displacement at the top was about 0.15 ft., with an average of about 0.03 ft. The displacement depends directly on the head of fresh concrete. Several factors, aside from stiffness, affect the movement of forms: (1) character of the interior bracing; (2) type of aqueduct; in "firm earth" type, there is no support of the outside forms in the lower portion; (3) weather conditions; in cool weather, concrete sets slowly, increasing the hydrostatic pressure of liquid concrete; (4) the length of arch concreted and the number of batches of concrete placed per hr.; the more rapidly the forms are filled, the greater will be the pressure and deformation; (5) the number, size, and position of stay-bolts between inside and outside forms. A light form, properly braced, gives a good aqueduct section, but will not stand handling.

The removal of the bulkhead of forms was expedited by some contractors on the Catskill aqueduct by the aid of a simple steam coil in the bulkhead, kept warm over night, thus accelerating the hardening of the concrete. Combined air and water jets under pressure were found very effective in cleaning forms. Steam jets hastened the drying of forms after cleaning. After the removal of all forms, all bolt holes through the concrete, if any, must be *thoroughly filled* with mortar; a little mortar plug will not do (see "Concrete Forms for Catskill Aqueduct," by Alfred D. Flinn, *J. A. Concrete Inst.*, June, 1915).

AQUEDUCT BRIDGES

Cabin John Bridge, a stone arch of 220 ft. span, carrying Washington aqueduct over the run, was built in 1863. The conduit is 9 ft. in diam. Owing to the extension of the reservoir system, the crown of the conduit is now under 2 ft. head. The original depth of water being 7 ft., the upper segment of the arch was not plastered. Leakage became so great in 1905 that the remainder of the joints were pointed, remedying 90 per cent. of the leakage. The remaining leakage came from temperature cracks caused by the unequal expansion of the two sides, the axis of the bridge lying nearly due east and west. These long temperature cracks lie in the lower quadrant of the conduit on north side of the bridge. The unequal expansion has opened the brick rings of the conduit along lengths of cracks, and the upper quadrant is moving over the lower at a rate of $\frac{1}{8}$ in. per yr. The remedy is a segmental cast-iron lining, 8 ft. 6 in. diam.[11]

Assabet Bridge, Wachusett aqueduct, has a length of 359 ft., composed of seven granite arches (one over road and six over a mill pond), with spans of 29.5 ft.; height of the crowns above pond, 17 ft. The lower half of the aqueduct on the bridge has the same form as at other places, but the upper part has vertical walls, roofed with I-beams and brick arches, instead of the semicircular arch. A lead lining was provided to prevent leakage from the aqueduct where it crosses the bridge, and it was absolutely watertight, winter and summer, for a number of years. The lead weighed 5 lb. per sq. ft.; came in sheets 16 ft. long and 9 ft. wide. Sheets were burned together without solder. To insure the permanence of the lead and make satisfactory work, the bottom and sides of the aqueduct, against which lead was to be placed, were smoothly plastered with $\frac{1}{2}$ in. of Portland cement mortar, and then coated with

asphalt. After the sheets of lead had been put in place, and burned together, they were covered with a thick coat of asphalt; then an 8-in. brick lining was put on. To protect the top of the bridge from weather, to prevent rain from soaking into the masonry, and to make a suitable finish, it was covered with granolithic, laid on two layers of roofing felt coated with coal tar.*

Wellesley Arches and Echo Bridge of the Sudbury aqueduct were lined similarly to the Assabet Bridge, but not for full height, as it was not feasible in the upper part. The remedy has been effective; it was applied after many years of annoyance from leakage and freezing. Echo Bridge, carrying Sudbury aqueduct across Charles River, leaked badly at haunches after 35 yr.; nine semicircular arches, 22 ft. 4 in. radius, carry brick aqueduct of horseshoe section, 9 by 7.5 ft. Cracks were found about 2 ft. either side of center line on invert, and directly over joints in roof stones of drainage gallery beneath. To remedy leaks, wetted perimeter was lined with sheet lead, $\frac{1}{16}$ to $\frac{1}{8}$ in. thick, weighing 3.5 lb. per sq. ft. Lead was procured in 14-ft. squares and connected by oxyhydrogen flame to form continuous lining. On invert, $1\frac{1}{2}$-in. floor was laid on lead, composed of 1 Portland cement to 2 parts pea gravel, reinforced with No. 26 expanded metal, finished with granolithic surface of 1:1 mortar, $\frac{1}{16}$ in. thick. Floor was vented by holes spaced 4 ft. apart, to prevent floating.[12] Up to 1925, the leakage has "not been troublesome."

Sunol aqueduct‡ is carried by a reinforced concrete arch of the open spandrel type.

AQUEDUCT TUNNELS

Shapes. Most contractors prefer to excavate a tunnel to rectangular shape; but horseshoe type is better adapted to withstand external heads. When driving a tunnel through rock of horizontal stratification, rectangular shape would leave a slab of rock self-supporting until lined, while horseshoe type would cantilever out horizontal layers in arched roof, giving more safety. For tunnels under hydraulic pressure, circular cross-sections are usually necessary.

Excavation Methods.† Methods have changed radically in last few years due to introduction of pneumatic hand drills. Formerly, when trimming had to be done by hand and was expensive, it was good practice to set heading holes well outside neat line to do away with trimming. With pneumatic hand drills, trimming is greatly cheapened; it is now good practice to set main holes closer and do a great deal more trimming, resulting in much more accurate excavation (diminished breakage), with correspondingly reduced quantities of masonry. In event of excess breakage, backfilling must be done with care to prevent subsequent ground movements. After 50 years, a cave-in occurred at Natick on Sudbury tunnel, presumably due to decay of the permanent timbering.[13]

Rate of progress varies with type of plant and geological conditions. On Catskill aqueduct, the average of all headings was: Rondout tunnel 183 ft.

* Leaks are not troublesome now (1925 Report), but additional waterproofing will be necessary later.

† For further discussion, see Peele's "Mining Engineer's Handbook" (Wiley, 1918), and other standard works.

‡ San Francisco.

per month in hard rock and 265 ft. in soft rock; Wallkill, 312 ft. in soft rock; Moodna, 193 ft. in hard, and 227 ft. in soft; City tunnels, 175 ft.

Shafts in Rock.* Circular shafts are no more difficult or expensive to sink than rectangular of same size. Long rectangular shafts are easier and cheaper to excavate, due to larger quantity which can be taken out as bench, except that corners are expensive and troublesome. Circular shafts are free from corner difficulty and, yard for yard, often cost no more than rectangular. Elliptical shafts are better than either, having advantages of both; are of good section to sink through soft, water-bearing ground as they resist hydrostatic pressure well. Circular shafts are best for waterways; they were used extensively on Catskill waterworks of New York City.

Grade Tunnels. *Ground-water Admission.* Catskill aqueduct grade tunnel linings are not designed to resist external water pressure; excepting at a few places where quality is objectionable, ground water is admitted freely by venting underdrains and by building weepers into side walls. Design is based on assumption that ground-water head will exceed head of water in aqueduct except at portals, where tunnels are made tight and stronger.

Pressure Tunnels. *Relation of Depth of Cover to Hydrostatic Pressure Head.* New Croton aqueduct, ratio of depth of cover (rock and earth) to pressure head varies from 0.572 to 0.703. On Catskill aqueduct, New York, minimum allowed rock cover is 150 ft.; maximum unbalanced bursting pressure, 420 ft.; maximum external hydrostatic pressure, tunnel empty, 1100 ft. (Hudson River crossing); maximum depth below hydraulic gradient, 1500 ft. (see Table 76).

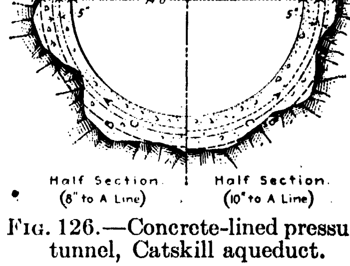


FIG. 126.—Concrete-lined pressur tunnel, Catskill aqueduct. (Diameters 11′ 0″ to 16′ 7″.)

Concrete lining, Catskill aqueduct, has various thicknesses, see Table 76, corresponding to various diameters and heads. These thicknesses represent the distance from waterway to "C" line. Rock is allowed to project 5 in. within this so-called "C" line (Fig. 126), but no timbering left in place can project within it. This "C" line was fixed 8 in. inside "B" line, the line about which the excavation is averaged. This "B" line was fixed from a study of breakage in existing tunnels as an average payment line. Contractor is not paid for excavation beyond this line; all excavation and concrete are measured within this line. Concreting was done in five stages: (1) Placing invert, about 5 ft. wide, upon which rails were laid for running the forms for remainder of circumference. (2) Placing walls of tunnel up to, or slightly above, center line. (3) Placing arch, excepting a 2.5-ft. key at crown, where concreting was most difficult. (4) Concreting key from end of form, or making closure by special device, Fig. 127, or other means. (5) Grouting the dry packing,

* For further information see "Practical Shaft Sinking," by Francis Donaldson (McGraw-Hill Book Company, Inc., 1910).

Table 75. Pressure Tunnels for Water

(Catskill Aqueduct pressure tunnels on p. 295)

Name	Location	Size, ft.†	Hydrostatic head, ft.	Minimum depth below ground surface, ft.	Lining: Kind	Lining: Thickness	Grouting	Geologic formations	Length, ft.	Shafts: No.	Shafts: Depth, ft.	Remarks
First Chicago*.......	Chicago, Lake Mich.	5	70	35(e)	Brick	2 rings 4" brick	No	Stiff blue clay	10,567	2	70 75	Gas explosions
First Cleveland*.....	Cleveland, Lake Erie	5	90	54(e)	Brick	9"	No	Blue clay, quicksand	6,662	2	67 90	Gas and quicksand caused 20° deflection horizontally
Second and third Chicago*	Chicago, Lake Mich.	7	70	35(e)	Brick	2 rings 4" brick	No	Stiff blue clay	10,460 10,908	2	75	These tunnels enter same crib as first tunnel
Mississippi River.....	Falls of St. Anthony	3.5 × 4	--	---	Concrete	------	--	Soft sandstone	2,300	1	75	35' high; tunnel filled with masonry to stop underground stream
Dorchester Bay.......	Boston, Mass.	7.5	157	118	Brick, concrete backing	12" brick	No	Clay, slate, conglomerate	7,000	3	157(b)	Water course, 2 mi. wide. Discharge for sewage
68th St. Intake*.....	Chicago, Lake Mich.	5 6 7	--	14(e)	Brick	12"	No	Clay	5,026 12,153 8,132	3	60(b)	Many deflections in alinement
New Croton Aq......	New York	12.25 10.25	437	39	Brick, concrete and rubble backing	3 or 4 rings brick	Yes	Gneiss, some limestone	36,400 ±	15	426(a) 139(b) 80(c)	Water course, 400' wide
Gould's Swamp Siphon	New Croton Aq., N. Y.	14.25	70	105	Brick, concrete and rubble backing	3 rings brick	Yes	Gneiss	1,169	3	119(a) 78(b) 54(c)	Part of downtake shaft on incline
Washington Aq.......	D. C.	8.21 × 9.88	174	46	Brick, rubble backing	3 rings brick	Yes	Schists	20,696	8	170(a) 123(b) 60(c)	500' cast-iron lining, 9' diam. No work between 1888 and 1895
Four-mile Intake*....	Chicago, Lake Mich.	8 6	77	40(c)	Brick	13"	No	Clay and quicksand	9,139 25,200	3	75(b)	Quicksand caused substitution of two 6' tunnels for 8' tunnels inshore; united into 8' tunnel
Vyrnwy (Liverpool Supply)	Mersey Riv., England	10	359	29(e)	Cast iron	1¼" plate, 6" flange	No	Gravel, sand and clay	810	2	53(b)	Water course, 805' wide. Shield used. Water pipes placed in passage
Second Cleveland....	Lake Erie	7	--	---	Brick	3 rings	--	Clay, quicksand	9,177	2	115 145	Gas and quicksand interfered. Detours made
Milwaukee Intake*...	Lake Mich.	7.5	115	97	Brick	4 rings	No	Very hard clay, gravel, boulders, quicksand	3,200	3	80(b)	Water 1640' from shore necessitated deflecting tunnel sideways and upward
Lake View Intake*....	Chicago, Lake Mich.	6	70	45	Brick	8"	No	Clay, rock	10,000	3	80(b)	Many deflections in alinement
Northeast Intake*.....	Chicago, Lake Mich	10 (external)	99	64(e)	Brick	18"	No	Clay, sand	14,025	3	99(b)	....

Table 75. Pressure Tunnels for Water.—(*Continued*)

Name	Location	Size, ft.†	Hydrostatic head, ft.	Minimum depth below ground surface, ft.	Lining: Kind	Lining: Thickness	Grouting	Geologic formations	Length, ft.	Shafts: No.	Shafts: Depth, ft.	Remarks
Third Cleveland*	Cleveland, Lake Erie	9	109	60(*e*)	Brick, masonry backing	13″ brick	No	Hard clay, gravel and sand, quicksand	26,040	4	122(*a*) 118(*b*) 112(*c*)	Several gas explosions; workmen killed
Cincinnati Intake*	Cincinnati, Ohio River	7	146	52	Vitrified brick	15″	....	Limestone, shale	1,426	2	149 143	Measurements refer to high water. 70′ range from low to high water
Waterworks Land Tunnel	Cincinnati, Ohio	7	160	80	Brick, concrete backing	2 rings brick, $8\frac{1}{2}$″	Yes	Limestone, interbedded with clay and shale	22,250	5	160(*a*) 132(*b*) 110(*c*)	Tunnel is from 300′ to 1000′ from Ohio River. See p. 294
Champion Mill*	Plainsdale, Mich.	5.5 × 6.5	55	30	None	....	No	Sandstone and conglomerate	1,015	1	45	Under Lake Superior. 3 fissures in rock. Drill holes pierce roof for water supply
Adventure Mill*	Lake Superior	7 × 5	56	35	None	....	No	Sandstone	887	1	93	Drill holes pierce roof for water supply
Kaw River	Kansas City, Kan.	7	132	72	Brick, concrete backing	9″ brick	No	Hard soapstone	1,125	2	161 113	....
Shawinigan Falls	Canada	13	—	—	Reinforced concrete	15″	....	Blue clay	1,000	1	....	Conducts water to turbines through 50′ ridge
Torresdale	Philadelphia	10.5	120	85	Brick, concrete backing	2 to 5 rings back	Yes	Gneiss	13,809	11	136(*a*) 112(*b*) 100(*c*)	....
Los Angeles	California	5	80	116	None	....	No	Solid rock	2,500	1	116	Infiltration gallery. Wells pierce roof
New Southwest*	Chicago, Lake Mich.	13.67×14	130	98	Concrete	12″	No	Limestone, clay and quicksand	12,180	3	150 103 98	....
Lake Erie	Buffalo, N. Y.	14.75×15	—	—	Concrete	18″ floor 24″ roof 11.5″	Yes	Seamy rock	6,000	2	75	....
Crib 5	Cleveland	10	110	50	Concrete blocks		....	Clay	16,100	1	....	Shield deviated 180 ft. from line and 6 ft. from grade
Wilson Ave.	Chicago	12 × 12 13 × 13	119	—	Concrete	12″	No	Limestone	31,200 5,200	5	....	Lining placed pneumatically
Linwood Ave.	Milwaukee	9	30	—	Concrete	....	Yes	Hard clay, quicksand	7,400	4	44 100 120	Daily tests for gas

* Not strictly pressure tunnels being intake tunnels pumped from shaft at one end.
(*a*) Max. depth. (*b*) Average depth. (*c*) Min. depth. (*e*) Measured from invert.
† If circular, the one figure gives diameter of waterway; if not circular, extreme horizontal and vertical dimensions of waterway are given.

if any, and all seams and voids behind the lining, first at low pressures and finally at high pressures. See "Grouting Operations, Catskill Water Supply," by Sanborn & Zipser, *T. A. S. C. E.*, Vol. 83, 1919–1920, p. 980. Concrete lining on St. Louis tunnel was placed by compressed air; any voids behind lining (detected by sounding with a hammer), were grouted.[15] The 3-mi. Cleveland tunnel (1916), is lined with segmental concrete blocks and grouted.[16]

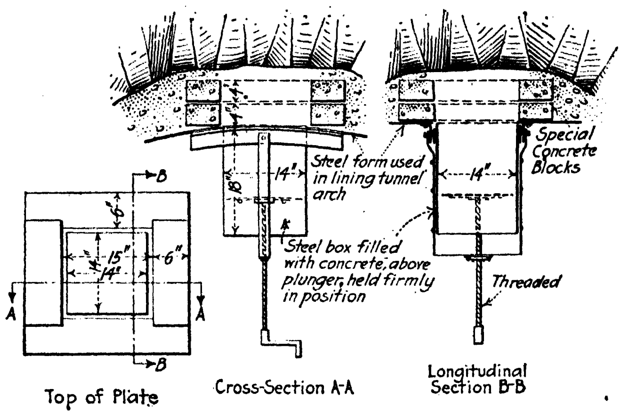


FIG. 127.—Catskill aqueduct. One method of placing last concrete in key of tunnel arch at a closure.

Leakage. *Cincinnati waterworks land tunnel* had a leakage inward of 10 gal. per min. for 22,250 ft., or 3400 gal. per day per mi.; later gagings showed 5800 gal. per day per mi., an increase of 70 per cent. in 8 months between the first and last gagings. The leakage concentrated in 1100 ft. of tunnel was at maximum rate of 117,500 gal. per day per mi. Leakage outward varied from 10 to 42 gal. per min.; maximum, 14,000 gal. per mi. per day, for the entire tunnel, or 290,000 gal. per day per mi. of the wet stretch. Maximum outward leakage occurred under a head of 34 ft.; inward, under an estimated head of 75 ft. The outward leakage of 10 gal. per min. was obtained under a head of 23 ft. for the first test, and 33 ft. for the fourteenth test, 1 month later. The reasons for the tightness of this tunnel are: (1) tightness of the rock, but 5 per cent. of which showed leakage before lining; (2) non-absorbent and doubtless impermeable character of the brick used; (3) the slow-setting mortar facilitated the complete filling of the joints; (4) exceedingly rigid inspection; (5) the large percentage of limestone dust probably made the concrete dense; (6) careful protection of the masonry from water during setting; (7) thorough grouting in wet ground.

In the *East Boston Subway tunnel*, a 1½-mi. stretch leaked 25 gal. per min. (24,000 gal. per day per mi.) before grouting; 7 gal. per min., or 6700 gal. per mi. per day after grouting.

Jersey City conduit, according to J. W. Hill[27] showed a leakage during construction of 134,000 gal. per day, or 154,000 gal. per day per mi., for Section 1, of which 1600 ft. was a tunnel through shale and sandstone, and 3000 ft. in open cut. The conduit is concrete-lined, with an internal diameter of 8 ft. 6 in.

Table 76. Pressure Tunnels of Catskill Aqueduct

Name	Location	Diam	Head, ft: Total on center line	Head, ft: Unbalanced on center line*	Min depth, ft below surface	Lining thickness†	Geology	Length, ft (horizontal)	Shafts: No.	Shafts: Depth, ft.: Max	Shafts: Depth, ft.: Av.	Shafts: Depth, ft.: Min.
Rondout	Catskill Aq north of N Y	14′ 6″	726	239	345	15″ and 17″	Shales, limestone, sandstone, quartzite	23,608	8	712	520	374
Wallkill	"	14′ 6″	544	279	265	15″ and 17″	Hudson River shales and sandstone	23,391	6	483	387	350
Moodna	"	14′ 2″	630	390	270	13″ and 15″	Shale and granite	25,200	7	586	451	342
Hudson	"	14′ 0″	1,517	417	1,100	13″ and 15″	Granite	3,022	2	1,185	1,172	1,159
Breakneck	"	14′ 0″	600	405	195	17″ and 19″	Granite	783	1		625	
Croton		14′ 0″	515	170	345	13″ and 15″	Granite	2,639	2	542	523	505
Yonkers		16′ 7″	155	55	100	15″ and 17″	Gneiss	12,978	4	155	130	106
City Tunnel‡	Catskill Aq in New York	15′ 0″	633	295	210	13″ and 15″	Gneiss and schist	40,760	11	474	308	216
		14′ 0″	653	262	208	13″ and 15″	Gneiss and schist	23,890	6	446	274	215
		13′ 0″	459	253	200	13″ and 15″	Gneiss and schist	6,840	2	223	218	213
		12′ 0″	988	252	204	11″ and 19″	Fordham gneiss, Manhattan schist, Inwood limestone,	10,550	2	741	730	708
		11′ 0″	1,005	295	308	11″ and 17″	Fordham gneiss, Manhattan schist, Inwood limestone	13,590	4	715	517	318
Schoharie	N of Ashokan	11′ 6″ × 10′ 3″	144	144	174	3 and 12″	Shale and sandstone	95,740	8	630	495	174

* Ground water assumed at surface
† See page 291
‡ See four articles by W. E. Spear, *E N.*, Jan. 14, 21, 28 and Feb 4, 1915.

The *New Croton aqueduct* tunnel had a leakage outward in 7 mi. under 130 ft. head of 225,000 gal. per day, or 32,100 gal. per day per mi. The leakage inward in 25 mi. of tunnel lined with brick (and concrete rubble in places) was 4 mgd. or 160,000 gal. per day per mi.

Catskill Aqueduct Tests. The cut-and-cover aqueduct between Fort Hill and Bryn Mawr siphons, 1.8 miles long, where conditions were not complicated by tunnel infiltration showed an outward leakage from 70,000 to 150,000 gals, per day.[28a]

Roundout pressure tunnel, when standing full, with water at el. 440, showed an outward leakage of 60 to 400 g.p.m. Wallkill pressure tunnel, with water at el. 440, showed an outward leakage of 25 to 300 g.p.m. Moodna-Hudson-Breakneck pressure tunnel, with water at el. 395, leaked outwardly 800 to 900 g.p.m.[28b]

City tunnel (pressure) between shafts 13 and 18, tested under pressure from Hillview reservoir, showed an outward leakage of 180 g.p.m. The portion south of Shaft 18, under same pressure, had an outward leakage of 662 g.p.m.[28c]

Croton Lake pressure tunnel, with water at hydraulic grade, had an outward leakage of 0.11 cfs. (99 g.p.m.) to 0.04 cfs. (36 g.p.m.) per mile of tunnel.[28d]

Silver Lake conduit, 4 ft. diam., 2800 ft. long, of which 1900 ft. had a reinforced concrete shell, under a pressure of 18 lbs. per sq. in., showed a final outward leakage of 9.2 g.p.m.[28e]

Hillview reservoir by-pass, under full reservoir head, showed an inward leakage of 104 g.p.m., in length of 2859 ft.[28f]

Tunnels: Capacity, Lined vs. Unlined.* With two rock tunnels of large size and same slope, one unlined would require twice the cross-sectional area of one lined with smooth masonry to carry same quantity of water.

Unlined portions of Wachusett rock (granite and diorite) tunnel have given little trouble since aqueduct has been in service (1898). No large masses have fallen, but small quantities of chips are found on invert each time aqueduct is cleaned. Loss of head due to roughness of rock is great.

AQUEDUCT MAINTENANCE

Fouling. Compensation must be made for the greater fouling of aqueducts near the head works. Observations show that a point exists, downstream from which fouling is practically constant. The fouling of Sudbury aqueduct extends downstream about 1000 ft., decreasing rapidly at first, and then more slowly; beyond 2 mi. from the head works, fouling is constant.†

Experience on Cochituate aqueduct, Boston (brick lining), is that deposits of slime and dirt allowed to accumulate give place to growths of spongilla, which after a year or two become hard, with long finger-like projections, forming serious obstructions to flow of water; maximum reduction of capacity was

* Experiments with a depth of 3 44 ft in Beacon St tunnel, Boston, where 4362 out of 4614 ft are without lining on the sides, show that, although the bottom is of concrete and reasonably smooth, the coefficient is 40 per cent smaller than for the brick conduit with the same hydraulic mean depth in the same formula. At no point is the rock excavation less than 1 ft. outside the lines of standard brick section. Yet it required a greater inclination of the water surface to flow the same volume as the brick conduit.[18]

† See Sudbury diagram, *E. N. R.*, June 30, 1921, p. 1132.

12.5 per cent. For Wachusett aqueduct,* Boston (brick lining below springing line, concrete above), 10 per cent. reduction in flow was measured at end of the first year, with coat of slime about $\frac{1}{16}$ in. thick, but no spongilla. New Croton aqueduct, New York (brick lining), after 9½ years' constant use without cleaning showed a reduction in flow of 14 per cent.

Freedom from Fouling of Concrete Lining. The deterioration in carrying capacity of concrete conduits is small, probably less than for brick. In 20 mi. of cement-lined pipe taken up in Northern New Jersey, no instance of growth or deposit was found, the pipe being apparently as clean and free from slime as on the day laid. An examination in 1911 of the Jersey City concrete aqueduct built in 1903 showed the same characteristic, the bottom and sides being free from slime.

Growths in Aqueducts and Canals. S. Fortier states that algæ cause losses in carrying capacity altogether disproportionate to the cross-sectional area occupied; algæ appear again shortly after being scraped off.† The Genus Cladophora, a long-fibered variety, causes most trouble in irrigation canals; it is said to be related to Conferva Bombycinum, which requires about 1 part copper sulfate to 1 million parts of water for extermination. This type does not seem to grow in darkness. In the concrete-lined canal of Arizona Power Co., tunnel portions were clean, while the canal portions contained algæ growths, although velocity was the same in both. Growth might be discouraged by painting concrete lining dull black,‡ for observations show more profuse growths on light surface than on dark. No ordinary stream velocity stops growths; algæ are not so numerous in silt-laden waters; water temperature seems to have little effect.[20] Organisms which lodge and develop in pipe may be animal or vegetable; they generally grow on the sides and the top, rarely on the bottom, as they appear to dislike the subsidence of fine silt apt to occur when the velocity is greatly reduced. Polyzoa, spongilla, algæ, and fungi are found in dark chambers of reservoir gate houses and in conduits and distributing pipes, but in the latter are usually restricted to the vicinity of the source or reservoir. In some cases, these growths are found in pipes several miles from the head.

Cleaning Aqueducts. Wachusett and Weston aqueducts were formerly cleaned about once a year, and Sudbury twice, restoring the flowing capacity to within 2 or 3 per cent. of that when first put into service; the decrease in capacity becomes about 10 per cent. by end of year.[21]

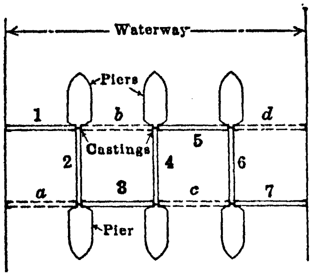
Unwatering aqueducts below the hydraulic gradient is accomplished by centrifugal pumps in floats, which pump their way down specially constructed drainage shafts,§ by sinking pumps used in mining, and by air-lift, as at St. Louis.[23]

AQUEDUCT APPURTENANCES‖

Plug Drain Valves for Chambers. For drainage, gate, and other chambers, 6-in. valves, with rubber plugs and rising stems have proved satisfactory;

* See Fig. 123, p. 283.
† C. W. Sherman dissents: "Organic growths—either fungi or animal forms classed as Protozoa—undoubtedly cause such losses, but not algæ."
‡ See p. 801.
§ See *E. N.*, June 4, 1914, p. 1244.
‖ See also p. 536.

iron guides should not be used for the plug; they rust so badly that the disk cannot be lifted. A non-rising stem necessitates a hole through disk, and if the screw threads do not fit perfectly, leakage results.



Full Lines show Normal Position.
Alternate Position, Screens in Bays
a, 2, b, 4, c, 6, d

FIG. 128.—Re-entrant bay arrangement of screens.

Screens* are necessary where a surface supply is used. Screens should be placed in chambers, giving access for removing accumulations before they become serious obstructions. Clean screens of ample area cause no loss of head. Clogged screens will be torn out if of fine wire. An automatic alarm to signal when screens become so clogged as to cause a loss of head which threatens to break them can be easily devised. A concrete-lined channel, 24 in. deep, at the bottom of screens serves as a sand trap. A general arrangement at headworks is: Guard the first inlets by outside racks of metal or wood bars, set edgewise 4 to 6 in. apart, to prevent larger objects from entering; inside the chamber, place coarse, copper screens, about 1-in. to 1½-in. mesh, to remove leaves and larger fish; finally guard entrances to pipes, pumps, or filters by screens having 4 to 6 meshes per in., arranged to be frequently removed and cleaned.

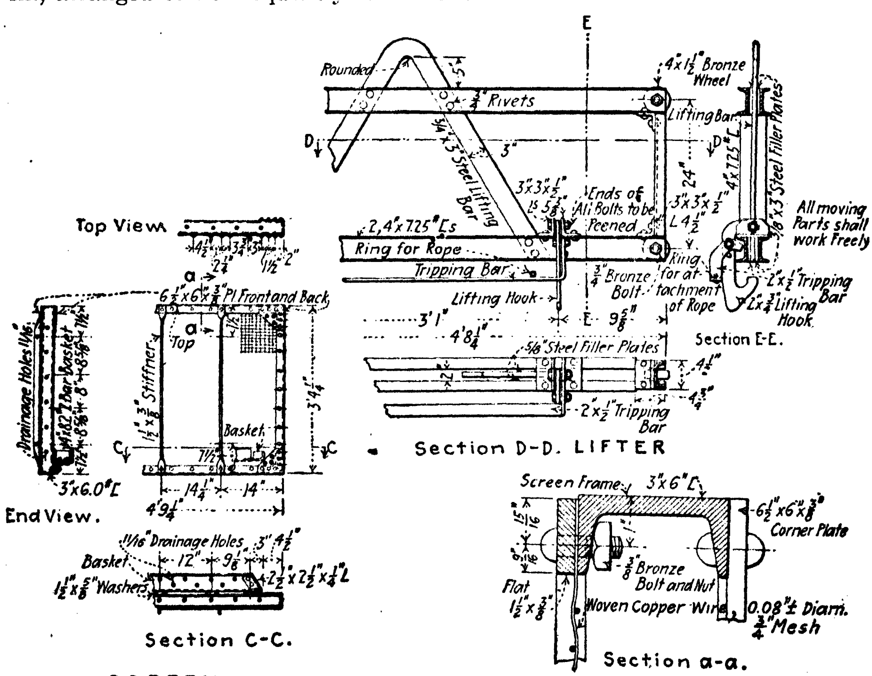


FIG. 129.—Catskill water supply, New York City. Screen and lifter.

Screens should be in duplicate in each passageway, where complete guard is required, so that one set can be placed before the other is removed for clean-

* See also Chap. 5, pp. 81, 82.

ing. Even with double screens, close together, with reasonable care in cleaning, some fish and trash will get into the pipes. The re-entrant bay arrangement of screens would not be suitable in many places, since the large spaces between screens put across the bays during cleaning and the regular screens, would contain fish and refuse that could not be conveniently removed. Inclined screens are objectionable since: (1) they cannot be used in deep water; (2) a large proportion of the fine dirt is rubbed through during cleaning, which is done by brushing.

Screen areas should be liberal. For screens guarding entrances to pipes, wire should be No. 14 or $\frac{1}{16}$ in. Design of screens and frames, and apparatus for lifting them (N. Y. Board of Water Supply, Fig. 129) has proved satisfactory. The Metropolitan Water Board standard screen is of copper (No. 16, Birmingham wire gage), of 6 meshes per in. Frames are of white pine, fastened with tree nails. The netting is stapled to the upstream side of the frame, and a $\frac{1}{2}$-in. board, full width of frame, is placed over the staples. A most important part of the equipment is the "dirt catcher"—a wire basket of the same netting as the screen, fastened to the bottom of the screen. Baskets were sometimes made detachable on Catskill aqueduct. Instead of wire staples, brass clips spaced 15 in. apart should be used to fasten the trash baskets to screen frames so that they can be removed easily.

Revolving screen, Roosevelt Canal,[24] was installed to remove floating débris at penstocks. Screen is in form of truncated cone, smaller end (7.5 ft. diam.) downstream; axis, (16.5 ft. long) inclined so that most of smaller end is above high-water line and over 50 per cent. of larger end (11 ft. diam.) is submerged. No. 18 galvanized wire, $\frac{1}{2}$ in. mesh, is supported by $\frac{5}{8}$-in. and $\frac{3}{4}$-in. circumferential rods, in turn supported by sixteen 4-in. by $\frac{3}{4}$-in. bars. Water enters at larger end which is free of interior bracing. Screen is held to position by an exterior trunnion ring, 5 ft. ± from larger end resting on trunnion either side of center line, and by a shaft fastened to smaller end. This shaft passes far enough into screen to support one end of collecting trough; other end slopes toward, and is supported by pier upstream from screen. Screen is turned $\frac{1}{60}$ r.p.m. through gearing and ratchet ring at upper end by 3-hp. motor. Area of submerged screen is 232 sq. ft., when canal is discharging 250 cfs. Revolutions carry débris to top of screen whence it drops into this trough. Periodical discharges from tank above screen wash out trough and free screen of any adhering débris.* Experience on St. Paul supply indicates advantage in cleaning and maintenance over the former stationary screens.[25] Denver also employs a revolving screen at the intake.[26]

Stop Planks. Grooves for stop planks must be smooth and straight. Instead of using single stop planks, fasten two or more together so as to form a shutter, which cannot come out of groove by tilting, while being raised or lowered. For great depths or large openings steel plate shutters with wooden bearing strips on edges are preferable to wood. Wooden stop planks should be weighted so as to just sink. They may be given a few coats of boiled linseed oil or creosoted. Long-leaf yellow pine is a very suitable wood. Stop-planks, Hinckley dam, N. Y. State Barge Canal, are built up of white-oak sticks, fastened between steel I-beams, in sections of convenient size.

*** Later advice shows that revolving screens may be not as economical as rackbar: Tank had insufficient capacity and elevation; men have to use rakes; grass and moss cling to screen.**

Each section is provided with bronze rollers to reduce sliding friction in both directions.

Floor covers over stop-plank and similar openings, if of wood, are easier to handle than if of metal, are fairly durable, and easy to replace, but if not frequently inspected, may become unsuspectedly weak, and so dangerous, from rotting on under side, while top remains sound. Rolled or pressed steel, or thin cast-iron plates are preferable. Reinforced concrete slabs may be used where they do not have to be moved often; they are heavy. Wrought-iron or steel plates should be heavily galvanized or kept well painted, particularly on under side.

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CHAPTER 15

PLATE METAL PIPES* (STEEL AND IRON)

Merits.† A steel pipe is more cheaply constructed in large sizes than is cast-iron pipe, and it can be installed more rapidly. It is more reliable than cast-iron in that it fails mostly by corrosion while a cast-iron pipe blows out at a weak point, causing larger damage to surrounding property. But a steel pipe is liable to float and will collapse under circumstances which would not affect a cast-iron line. Proper appurtenances must be provided against such mishaps.[1] A steel pipe can be tested in cold weather, when it would be inexpedient to test a cast-iron line because of possibility of breakages. Baltimore high-pressure fire system is constructed of steel pipe; no breaks in 10 years.[57] A steel pipe offers less resistance than cast-iron pipe to electrolytic action and soil corrosion.

RIVETED JOINTS

Longitudinal Seams. With low heads or pressures, longitudinal seams proportioned for tightness are of ample strength. Make rivet pitch of circular and longitudinal lap seams the same, as a few more rivets will not increase the cost. Use lap joint up to $\frac{5}{8}$-in. plate, above which a good job is difficult; butt joints result in stronger pipe. One or two longitudinal seams are used, depending on diam. and thickness of pipe. Double riveting for pipe above 48-in. diam. is common practice.

Circular Seams.‡ Make number of rivets a multiple of 4, so that they will be at 90° points. Single riveting is generally strong enough. Either lap or butt joints are acceptable to shops, no matter what the thickness of plate, the lap joint being easier to make. The question of circular seams is an economic one, based on gain in hydraulic properties, the butt joint providing the smoother waterway, but adding from 0.5 to 0.8 cts. per lb. to cost. See p. 356 for temperature stresses.

Lap-riveted Joints. Lap length is reckoned from edge to edge of bevel. Length of lap is determined by the shearing value of the rivet through the plate, and should not be less than $T \times$ the effective area of plate between rivets, $\div S$; T = strength in tension, S = strength in shear. Boiler makers commonly assume the lap as 3 diam., probably because of wedge action of rivet. A single-riveted joint may fail by deficient plate bearing, shearing rivets, tearing of plate, rivet shearing through plate, and rivet breaking through plate. The breaking of rivet through plate depends on lap and pitch and is difficult to calculate; it need not be considered in most cases, as the pitch must be close

* Manufacturers include Biggs Boiler Works Co., Akron, O.; East Jersey Pipe Co., Pittsburgh Penstock Construction Co., Sharon; Petroleum Iron Works Co., Sharon, Riter-Conley Co., Pittsburgh.
† See also pp. 359 and 399.
‡ Called also "girth seams."

to insure tight work, and only where large pitches are used need breaking be considered.

Butt Joints. Butt straps require double the riveting and have double the chance to leak both at rivets and edges, to say nothing of the increased work and materials, but are necessary for strength in many cases. Double-riveted lap and double-riveted butt joints have the same strength; but the lap joint is cheaper. A joint having a double-riveted butt strap on the outside and a triple-riveted butt strap on the inside can be designed to have efficiency of about 85 per cent.

Rivet holes should be $\frac{1}{16}$ in. greater than the nominal diam. of the rivet; the driven rivet should fill the hole. Specifications commonly forbid drifting; some experienced pipe makers believe that, with good plate, drifting to the extent of $\frac{1}{16}$ or even $\frac{1}{8}$ in. is not objectionable. Plates should be punched from the contact side. Catskill aqueduct specifications are as follows:

Rivet holes shall be spaced with precision. Holes in $\frac{1}{2}$-in. and thicker plates shall be drilled from solid, or punched to approved size and reamed. Punched holes shall be clean cut, without torn or ragged edges, and in punching only the best and sharpest punches and dies shall be used. All burrs or splits caused by punching shall be removed by suitable means. Corresponding holes shall, without enlarging, coincide to within about $\frac{1}{32}$ in., and all plates in which corresponding holes do not so coincide shall be rejected, unless the engineer shall be of opinion that conditions of stress are such at the point in question as to make more extensive enlargement safe. Drift pins shall not be used in forcing holes, but any perceptible lack of coincidence in acceptable plates shall be corrected by a sharp reamer or drill.

Rivets. There is little if any difference in driving between soft steel and iron rivets, but steel rivets are better; the softer rivet has the lower shearing strength. The size of rivet for theoretical efficiency varies with type of joint and thickness of plate, being usually for lap joints (with rivets, $\frac{5}{8}$ to $1\frac{1}{8}$ in.) about twice the plate thickness. Maximum economy results when crushing strength equals shear. On Catskill aqueduct, the use of one size rivet (1 in.) throughout proved more economical, as the increased thickness of cover plate more than offset the decreased thickness of the main plate, in the butt joints (pipes 9- to 11-ft. diam.). For thick plates ($\frac{3}{4}$ in. and greater) $1\frac{1}{8}$-in. rivets are preferred, since small rivets cool before being thoroughly driven. Practice is divided on hot and cold rivets. The pinching of a hot rivet in cooling adds to strength as well as to tightness. Cold rivets are sometimes used for small and light work, and at least one large shop uses them regularly for large work—has used them successfully for some of the heaviest pipe made. With cold-driven rivets there is no layer of slag between rivet and plate; there is less lateral contraction and so a closer fit in hole. The principal danger, if the pressure of the riveting machine is not applied at right angles to the plate, is that the head may be partly sheared off, but such defects are easily discovered, especially during the hydrostatic test, when such heads come off. This should not be considered characteristic of cold-driven rivets, but merely accidental. Some makers can give lower price if cold rivets are used. Driving cold requires four or five times as much power.

Table 77. Hydrostatic Pressures for Driving Hot Rivets
(R. D. Wood & Co.)

Rivet diam., in.	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
Pressure, tons..................	20	25	33	45	60	75	100	125	150

Shop Riveting. Table 77 is based on the rivet passing through only two plates, the combined thickness of which does not exceed the rivet diam.

Another manufacturer uses a driving pressure of 63 tons for a 1-in. rivet, irrespective of thickness of plate; this brings the plates together well; a higher pressure would tend to deform the plates.

Rivet heads may be safely flattened to a thickness equal to one-fourth the diam. of the rivet. The splitting of rivet heads depends on length of rivets, not diam., and is due to contraction in cooling. Full rivet heads can be tightened to prevent leakage, by applying pressure to the cold rivet. On Catskill aqueduct, rivet calking was not allowed if the rivet was loose, unless the leak was slight and easily closed, otherwise the rivet was cut out and a new one headed up tight while hot. It is difficult to make countersunk rivets tight, even by the best means yet devised; a number will leak in the first instance. Calking of outside heads onlp can be done, and if this does not stop leakage the rivet must be cut out (requiring the emptying of the pipe). To countersink rivets on the inside (to improve hydraulic qualities) adds less than $\frac{1}{4}$ ct. per lb. to the pipe cost. Recent Brooklyn conduits had flattened heads.

All riveting in longitudinal seams should be done by hydraulic, compressed-air, or steam machinery. The pressure is retained until the rivet head has been perfectly formed and the metal has lost its red color. Suitably applied pressure holds the plates in close contact until the rivet is driven. Rivets should be driven at such heat as to give the best results.

Pitch of Rivets. Single and double riveting are used for lap, and double and triple riveting for butt joints. Double riveting is 16 to 20 per cent. stronger than single. *Joint efficiency* is the ratio of stress to strength, using unit stresses based on tests With $\frac{1}{2}$, $\frac{7}{16}$, and $\frac{9}{16}$ plates on the Catskill aqueduct, an edge distance of $1\frac{5}{8}$ in. was used with 1-in. rivets. In single-riveted lap joints, the minimum pitch of rivets depends on the value of tearing of the plate. The maximum pitch is determined by requirements of calking. One manufacturer has the general rule that the pitch of rivets in butt joints should not exceed eight times the thickness of the thinnest butt plate. Standards of Pacific Coast Electrical Assn. are as follows: Maximum pitch along calked edges $= 2\frac{1}{2}t +$ rivet diam. $+ 1\frac{1}{2}$. The minimum longitudinal pitch in a single-riveted lap joint is generally limited by the clearance between rivet heads, usually taken as $\frac{1}{2}$ in.

Table 78. Rivet Gages and Edge Distances for Steel Pipes

Rivet diam., in.	Center of rivet to edge of plate, in.		Spacing of rivet lines, in.	
	1	2	1	2
$\frac{1}{2}$	$\frac{27}{32}$	1	$1\frac{5}{16}$	$1\frac{1}{2}$
$\frac{5}{8}$	$1\frac{1}{32}$	$1\frac{1}{8}$	$1\frac{15}{32}$	$1\frac{3}{4}$
$\frac{3}{4}$	$1\frac{7}{32}$	$1\frac{1}{16}$	$1\frac{5}{8}$	2
$\frac{7}{8}$	$1\frac{13}{32}$	$1\frac{1}{2}$	$1\frac{7}{8}$	$2\frac{1}{2}$

1. Practice recommended by Hartford Steam Boiler Inspection & Insurance Co.
2. Practice of Chicago Bridge & Iron Works.

Table 79. Approximate Finished Weight of Steel Pipe
Pounds per lin. ft., including laps, bars, rivets, and coating*

Kind	Diam., in.	Thickness, in. $\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$
Double-riveted longitudinal joints	20	52.50	70.20	89.20	108.10	125.98	145.10
	22	57.22	76.45	96.55	117.25	136.89	157.70
	24	61.95	82.70	103.90	126.40	147.80	170.30
	26	66.61	88.98	111.15	134.81	158.71	181.61
	28	71.27	95.26	118.40	143.22	169.62	192.92
	30	76.00	101.55	125.65	151.65	180.55	204.25
	32	80.90	107.73	133.23	160.66	191.53	216.00
	34	85.80	113.91	140.81	169.67	202.51	227.75
	36	90.70	120.10	148.40	178.70	213.50	239.50
	38	95.88	126.61	155.81	188.50	224.62	252.08
	40	101.06	133.12	163.22	198.30	235.71	264.66
	42	106.25	139.65	170.65	208.05	246.85	277.25
	44	111.03	145.76	179.63	217.50	256.90	289.83
	46	115.81	151.87	188.61	226.45	266.95	302.41
	48	120.60	158.00	197.60	236.40	277.00	315.00
	50	124.99	164.15	205.31	246.08	288.61	326.85
	52	129.38	170.30	213.02	255.76	300.22	338.70
	54	133.75	176.45	220.75	265.45	311.85	350.55
	56	138.46	182.86	228.13	274.56	322.63	363.76
	58	143.17	189.27	235.51	283.67	333.41	374.97
	60	147.90	195.70	242.90	292.80	344.20	387.20
	62	153.70	202.30	251.65	302.28	355.08	399.25
	64	159.50	208.90	260.40	311.76	365.96	411.30
	66	165.25	215.50	269.15	321.25	376.85	423.35
	68	171.10	222.17	277.20	330.70	387.43	435.70
	70	176.90	228.84	285.25	340.15	398.01	448.05
	72	182.70	235.50	293.30	349.60	408.60	460.40
Double lock-bar pipe	20	57.03	74.00	91.30	113.50	137.30	150.80
	22	61.74	79.62	98.10	122.06	146.70	162.50
	24	66.46	85.24	104.85	130.60	156.00	174.40
	26	71.18	91.32	112.47	138.90	165.60	185.37
	28	75.90	97.23	119.85	147.52	175.20	196.34
	30	80.64	102.95	126.96	156.47	184.81	207.31
	32	85.11	108.87	134.38	165.11	195.22	219.27
	34	89.58	114.98	141.95	173.43	205.88	231.23
	36	94.06	121.25	149.60	181.40	216.72	243.20
	38	98.43	127.83	157.82	191.72	226.32	255.87
	40	102.80	134.19	165.45	200.67	236.94	268.54
	42	107.18	140.30	172.49	208.29	248.47	281.22
	44	112.67	147.33	181.57	218.59	259.18	291.69
	46	118.16	154.36	190.65	228.89	269.89	302.16
	48	123.67	161.40	199.72	239.20	280.60	312.60
	50	....	168.01	206.53	248.13	290.94	324.61
	52	....	174.62	213.34	257.06	301.28	336.62
	54	....	181.25	220.15	266.00	311.62	348.65
	56	....	187.52	228.78	274.97	323.24	361.93
	58	....	193.79	237.41	283.94	334.86	375.21
	60	....	200.05	246.05	292.90	346.50	388.50
	62	....	208.00	254.34	307.28	357.84	401.50
	64	....	215.95	262.63	317.60	369.18	414.50
	66	....	223.91	270.94	323.86	380.52	427.52
	68	....	231.19	280.23	333.83	391.68	439.52
	70	....	238.47	289.52	343.80	402.84	451.52
	72	....	245.76	298.81	353.76	414.00	463.53

* From "Handbook of Pipe" (East Jersey Pipe Co.). Approximate formula: Weight in lb. per lin. ft. = (Diam. in in. × Thickness in in. × 100 ÷ 8) + 10.

Costs depend on many conditions. Pipes with tapered, or "stove-pipe," joints cost a little more than alternate inside and outside courses, and butt-strap joints much more, especially if the inside rivet heads are countersunk. Tapered rings, or lengths, require lay-out plates and curving the plates to a cone. Recent riveted conduits in Brooklyn are required to be tapered.[57] All types of pipes are now laid with tapered rings, the interior seams pointing in

Table 80. Double-riveted Steel Hydraulic Pipe*

Diam., in.	Thickness: U. S. wire gage	Thickness, In.: Common fractions	Thickness, In.: Decimals	Head, ft., pipe will safely stand ‡	Weight† per lin. ft., lb.
①	②	③	④	⑤	⑥
3	18	1/20	0.05	810	2.3
4	18	1/20	0.05	607	3.0
4	16	1/16	0.06	760	3.8
5	18	1/20	0.05	485	3.8
5	16	1/16	0.06	605	4.5
5	14	5/64	0.08	757	5.8
6	18	1/20	0.05	405	4.3
6	16	1/16	0.06	505	5.3
6	14	5/64	0.08	630	6.5
7	18	1/20	0.05	346	4.8
7	16	1/16	0.06	433	6.0
7	14	5/64	0.08	540	7.5
8	16	1/16	0.06	378	7.0
8	14	5/64	0.08	472	8.8
8	12	7/64	0.11	660	12.0
9	16	1/16	0.06	336	7.5
9	14	5/64	0.08	420	9.3
9	12	7/64	0.11	587	12.8
10	16	1/16	0.06	307	8.3
10	14	5/64	0.08	378	10.3
10	12	7/64	0.11	530	14.3
10	11	1/8	0.13	607	16.3
10	10	9/64	0.14	680	18.3
11	16	1/16	0.06	275	9.0
11	14	5/64	0.08	344	11.0
11	12	7/64	0.11	480	15.3
11	11	1/8	0.13	553	17.5
11	10	9/64	0.14	617	19.5
12	16	1/16	0.06	252	10.0
12	14	5/64	0.08	316	12.3
12	12	7/64	0.11	442	17.0
12	11	1/8	0.13	506	19.5
12	10	9/64	0.14	577	21.8
13	16	1/16	0.06	233	10.5
13	14	5/64	0.08	291	13.0
13	12	7/64	0.11	407	18.0
13	11	1/8	0.13	467	20.5
13	10	9/64	0.14	522	23.0
14	16	1/16	0.06	216	11.3
14	14	5/64	0.08	271	14.0
14	12	7/64	0.11	378	19.5
14	11	1/8	0.13	433	22.3
14	10	9/64	0.14	485	25.0
15	16	1/16	0.06	202	11.8
15	14	5/64	0.08	252	14.8
15	12	7/64	0.11	352	20.5
15	11	1/8	0.13	405	23.3
15	10	9/64	0.14	453	26.0
16	16	1/16	0.06	190	13.0
16	14	5/64	0.08	237	16.0
16	12	7/64	0.11	332	22.3
16	11	1/8	0.13	379	24.5
16	10	9/64	0.14	425	28.5
18	16	1/16	0.06	168	14.8
18	14	5/64	0.08	210	18.5

Diam., in.	Thickness: U. S. wire gage	Thickness, In.: Common fractions	Thickness, In.: Decimals	Head, ft., pipe will safely stand ‡	Weight per lin. ft.,† lb.
①	②	③	④	⑤	⑥
18	12	7/64	0.11	295	25.3
18	11	1/8	0.13	337	29.0
18	10	9/64	0.14	378	32.5
18	8	11/64	0.17	460	40.0
20	16	1/16	0.06	151	16.0
20	14	5/64	0.08	189	19.8
20	12	7/64	0.11	265	27.5
20	11	1/8	0.13	304	31.5
20	10	9/64	0.14	340	35.0
20	8	11/64	0.17	415	45.5
22	16	1/16	0.06	138	17.8
22	14	5/64	0.08	172	22.0
22	12	7/64	0.11	240	30.5
22	11	1/8	0.13	276	34.5
22	10	9/64	0.14	309	39.0
22	8	11/64	0.17	376	50.0
24	14	5/64	0.08	158	23.8
24	12	7/64	0.11	220	32.0
24	11	1/8	0.13	253	37.5
24	10	9/64	0.14	283	42.0
24	8	11/64	0.17	346	50.0
24	6	13/64	0.20	405	59.0
26	14	5/64	0.08	145	25.5
26	12	7/64	0.11	203	35.5
26	11	1/8	0.13	233	39.5
26	10	9/64	0.14	261	44.3
26	8	11/64	0.17	319	54.0
26	6	13/64	0.20	373	64.0
28	14	5/64	0.08	135	27.3
28	12	7/64	0.11	188	38.0
28	11	1/8	0.13	216	42.3
28	10	9/64	0.14	242	47.5
28	8	11/64	0.17	295	58.0
28	6	13/64	0.20	346	69.0
30	12	7/64	0.11	176	39.5
30	11	1/8	0.13	202	45.0
30	10	9/64	0.14	226	50.5
30	8	11/64	0.17	276	61.8
30	6	13/64	0.20	323	73.0
30		1/4	0.25	404	90.0
36	11	1/8	0.13	168	54.0
36	10	9/64	0.14	189	60.5
36		3/16	0.19	252	81.0
36		1/4	0.25	337	109.0
36		5/16	0.31	420	135.0
40	10	9/64	0.14	170	67.5
40		3/16	0.19	226	90.0
40		1/4	0.25	303	120.0
40		5/16	0.31	378	150.0
40		3/8	0.38	455	180.0
42	10	9/64	0.14	162	71.0
42		3/16	0.19	216	94.5
42		1/4	0.25	289	126.0
42		5/16	0.31	360	158.0
42		3/8	0.38	435	190.0

Table 80 was computed by Byron Jackson Iron Works, Inc.

* Unit stress = 15,000 lb. per sq. in.; joint efficiency = 71 per cent.
† Including laps and rivet heads.
‡ No allowance for water-hammer or corrosion.

direction of flow, *i.e.*, the smaller diam. is laid downstream. Studies indicate that frictional resistances in 48-in. pipe are slightly less with tapered than with in-and-out courses. The price for steel pipe can be reduced several per cent. if liberal time be allowed for shop work.

Shop Calking. Lap joints are calked all around, inside and outside; butt joints on the outside only. See also p. 328 for Field Calking.

Shop Testing. To be sure of tight work, every shop length should be tested before coating. This can be done rapidly and cheaply by means of special heads with rubber or similar gaskets, requiring no riveting. Use flat-head, soft-rubber plugs in empty rivet holes. Same test heads can be used in field tests. One advantage of shop test is that refilling of the trench between field joints can be done at once, thus protecting the pipe from extreme expansion and contraction. It is cheaper to make the pipe tight in the shop than in the field. Some shops are equipped with testing machines.

DESIGN OF RIVETED JOINTS, STEEL-PIPE SIPHONS, CATSKILL AQUEDUCT*

Assumptions. No allowance was made for corrosion with mortar-lined and concrete-covered pipes and the nominal was taken as the working thickness of the steel plate. With unlined pipes, the working thickness of steel plate was assumed $t - \frac{1}{16}$ in. All units are in inches, or lb. per sq. in.

a_1 length of plate between rivet holes $= p_1 - d_1$.
d = nominal diam. of rivet.
d_1 = driven diam. of rivet $= d + \frac{1}{16}$.
ϵ = efficiency of joint.
f_c = unit bearing stress on rivets and plates.
f_s = unit shearing stress on plate.
f_s' = unit shearing stress on rivets in single shear.
f_s'' = unit shearing stress on rivets in double shear.
f_t = unit tensile stress on steel plate.
g = gage distance (*i.e.*, distance between two rows of rivets).
h = edge distance, from center of rivet.
h' = edge distance, from edge of rivet hole.
k = allowance for clearance between rivet heads $= \frac{1}{2}$ in.
p diagonal pitch of rivets.
p_1 longitudinal pitch of rivets.
t thickness of steel plate.

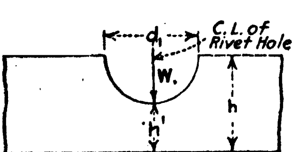


Fig. 130.—Steel pipe, edge distance.

Working stresses assumed were $f_t = 15{,}000$; $f_c = 22{,}500$; $f_s = 10{,}000$; $f_s' = 9000$; $f_s'' = 8000$.

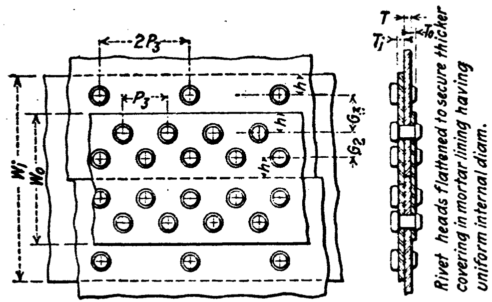
Edge Distance. Theoretical edge distance:

$$h = d_1\left(0.5 + 0.595\sqrt{\frac{d_1}{t}}\right).$$

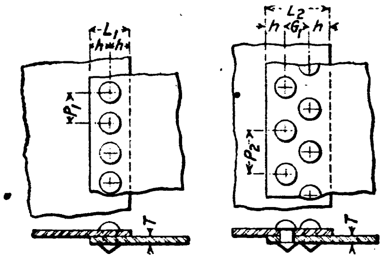
Practical edge distance: $h = 1.5d_1$.

Gage Distance and Diagonal Pitch. I. The gage distance, g, required for equal diagonal and longitudinal strength, *i.e.*, make resistance to tear and shear along $k - f$ equal to one-half resistance to tear along $n - e$ (see Fig. 132). To find g when p_1, d_1 and t are known. Resistance to rupture along $k' - f'$

* Following formulas, tables, etc. are given as examples only.



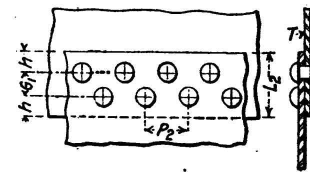
Longitudinal Seam Triple - Riveted Butt.
(81% Efficiency)



Single-Riveted Lap.
(50% Efficiency)

Double-Riveted Lap.

Circular Seam.



Longitudinal Seam.
Double-Riveted Lap.
(70% Efficiency)

Thickness of Shell	Longitudinal Seam			
	Pitch of Rivets	Gage Distance	Edge Distance	Lap
T	P_2	G_1	h	L_2
7/16"	3.50"	1 15/16"	1 5/8"	5 3/16"
1/2"	3.11"	2"	1 5/8"	5 1/4"

All pipe to have alternate inside and outside courses (i.e. not taper).
All rivets 1 in. diam.
Inside diameter of inside course = 9'6"

Thickness of Shell	Longitudinal Seam								
	Pitch of Rivets		Gage Distance		Edge Distance	Width of Cover Plates		Thickness of Cover Plates	
	Inner Rows	Outer Rows	Inner Rows	Outer Rows		Outer	Inner	Outer	Inner
T	P_3	$2P_3$	G_2	G_3	h	W_o	W_i	T_o	T_i
1/2"	3.58"	7.16"	1 15/16"	3 1/8"	1 5/8"	10 3/8"	16 5/8"	7/16"	9/16"
9/16"	4.00"	8.00"	2 1/16"	3 1/8"	1 5/8"	10 5/8"	16 7/8"	1/2"	5/8"
11/16"	3.82"	7.64"	2 1/16"	3 1/8"	1 5/8"	10 5/8"	16 7/8"	1/2"	6/8"
3/4"	3.58"	7.16"	1 15/16"	3 1/8"	1 5/8"	10 3/8"	16 5/8"	1/2"	5/8"

Thickness of Shell	Edge Distance	Single Riveted			Double Riveted			
		Number of Riv.	Approx. Pitch	Lap	No. of Rivets each Row	Approx. Pitch	Gage Distance	Lap
T	h		P_1	L_1		P_2	G_1	L_2
7/16"	1 5/8"	144	2.5"	3 1/4"	103	3.50"	1 15/16"	5 3/16"
1/2"	1 5/8"	Not Used			116	3.11"	2"	5 1/4"
9/16"	1 5/8"	"	"		123	2.95"	2 1/16"	5 5/16"
11/16"	1 5/8"	"	"		139	2.61"	2 3/16"	5 7/16"
3/4"	1 5/8"	"	"		145	2.50"	2 3/16"	5 7/16"

Assumed tensile stress on net section of plate 15,000 lbs. per sq. in.
" bearing " " rivets and plate 22,500 " " " "
" shearing " " plate 10,000 " " " "
" " " " rivets-single 9,000 " " " "
" " " " rivets-double 8,000 " " " "
All Computations based on diam. of driven rivets - 1 1/16"

FIG. 131.—Steel pipe siphons, riveted joints, Catskill aqueduct.

is 96 per cent. that along $k - f$,* and is a minimum for the pitch and gage distance assumed. Therefore resistance along $k - f$ must be made equal to $\frac{100}{96} \cdot \frac{a_1}{2}$

$$\left(1 - \frac{2d_1}{\sqrt{p_1^2 + 4g^2}}\right)^2 - \frac{\left[\frac{300}{96}(p_1 - d_1)\right]^2}{9p_1^2 + 16g^2} = 0.$$

Find the value of g to satisfy this equation for equal diagonal and longitudinal strength.

II. Gage distance for clearance of $\frac{1}{2}''$. Diam. of rivet head $= 2\left(d_1 - \frac{1}{16}''\right) = 2d$.

Hence

$$p = 2\left(d_1 - \tfrac{1}{16}''\right) + k = 2d + \tfrac{1}{2}''$$

$$g = \sqrt{p^2 - \left(\frac{p_1}{2}\right)^2} = \sqrt{\left(2d + \tfrac{1}{2}''\right)^2 - \left(\frac{p_1}{2}\right)^2}.$$

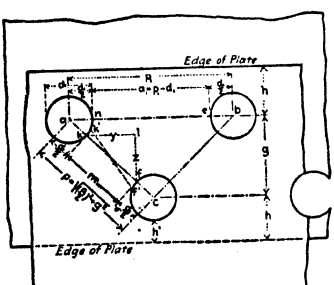


FIG. 132.—Gage distance and diagonal pitch.

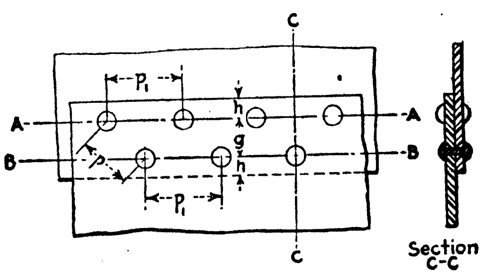


FIG. 133.—Double-riveted lap joint.

III. Gage distance for $\frac{1}{4}''$ greater net diagonal than longitudinal distance, *i.e.*, $2p = p_1 + d_1 + \frac{1}{4}''$

$$p = \frac{p_1 + d_1}{2} + \tfrac{1}{8}'' \qquad g = \sqrt{\left(\frac{p_1 + d_1}{2} + \tfrac{1}{8}''\right)^2 - \left(\frac{p_1}{2}\right)^2}.$$

Single-riveted Lap Joint (see p. 301). Length of joint $= p_1$. Methods of failure: (*a*) tearing plate; (*b*) shearing rivets.

Resistance, to tearing $= (p_1 - d_1)tf_t$; to shear $= \frac{\pi d_1^2}{4} f_s'$; of solid plate $= p_1tf_t$.

For *equal strength against tearing and shearing*, $p_1 = \frac{d_1}{1 - \epsilon}$ unless limited by clearance, when $p \nless 2d + \frac{1}{2}$ in. $\epsilon = \left(0.471\frac{d_1}{t}\right) \div \left(1 + 0.471\frac{d_1}{t}\right)$. The constant in the ϵ formula depends on the value of f_s' and f_t used, 0.471 being replaced by $\frac{\pi}{4} \times \frac{f_s'}{f_t}$.

Double-riveted Lap Joint. Length of joint $= p_1$. Methods of failure: (*a*) tearing plate along AA or BB; (*b*) shearing all rivets.

* Known by test.

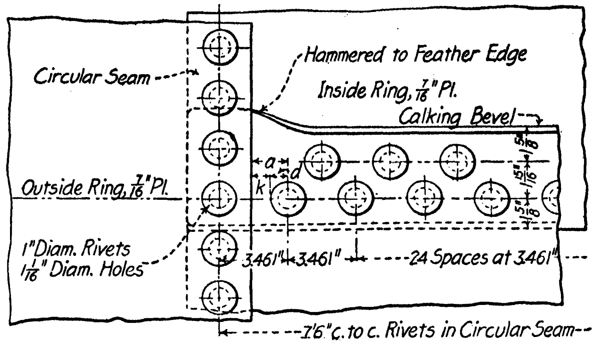


FIG. 134.—Junction of double-riveted lap joint (longitudinal seam) with single-riveted lap joint (circular seam). (Catskill aqueduct.)

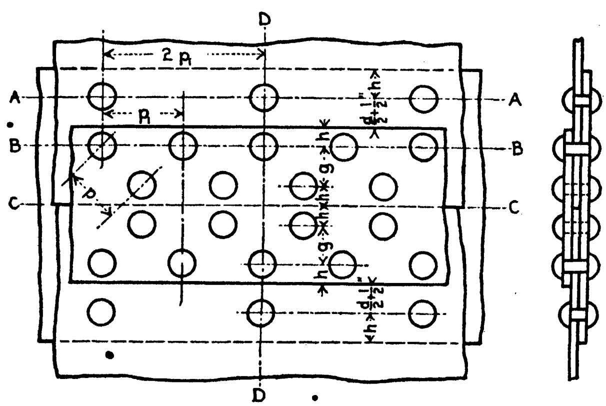


FIG. 135.—Triple-riveted butt joint. (See also p. 310.)

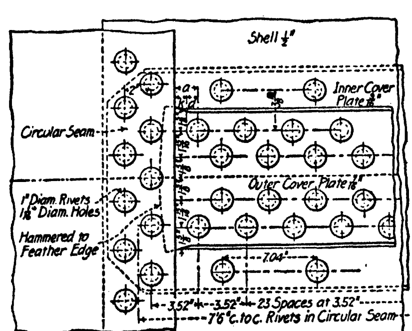


FIG. 136.—Junction of triple-riveted butt joint (longitudinal seam) with double-riveted lap joint (circular seam). (Catskill aqueduct.)

Resistance, to tearing along AA or $BB = (p_1 - d_1)tf_t$; to shearing $= \frac{2\pi d_1^2}{4}f_s' = \frac{\pi d_1^2}{2}f_s'$; of solid plate $= p_1tf_t$.

$p_1 = \frac{d_1}{1-\epsilon}$ unless limited by clearance, when $p \nless 2d + \frac{1}{2}''$

$\epsilon = 0.943\frac{d_1}{t} \div \left(1 + 0.943\frac{d_1}{t}\right)$. The constant 0.943 is for $f_t' = 15{,}000$, $f_s' = 9000$; for other values it becomes $\frac{\pi}{4}\frac{f_s'}{f_t}$. When $\epsilon = 70$ per cent., $d_1 = 2.472t$, $p_1 = \frac{d_1}{0.3}$.

Triple-riveted Butt Joint. Length of joint $= 2p_1$. Methods of failure: (a) tearing plate along AA; (b) tearing plate along BB and shearing rivets on AA; (c) shearing all rivets on one side of CC.

Resistance to tension $= tf_t(2p_1 - d_1)$.

Resistance to tension + shear $= \frac{\pi d_1^2}{4}f_s' + 2(p_1 - d_1)f_tt$.

Resistance to shear $= 8\left(\frac{\pi d_1^2}{4}\right)f''_s + \left(\frac{\pi d_1^2}{4}\right)f_s' = \frac{\pi d_1^2}{4}(8f''_s + f_s')$.

Resistance of solid plate $= 2p_1tf_t$.

PLATES FOR STEEL PIPES*

Materials. Catskill aqueduct specifications call for steel to be made by basic open-hearth process, plate steel ($\frac{7}{16}$, $\frac{1}{2}$, and $\frac{9}{16}$ in. thickness, for 5-ft. 6-in. to 11-ft. 3-in. diam.) to conform to "flange steel" and rivets to "boiler rivet steel" requirements of A. S. T. M., Serial designation A 30–21 and A 31–14, respectively.

Thick Plates. Most shops cannot handle plates of greater thickness than 1 in. (one can take $1\frac{1}{2}$ in.); $1\frac{1}{2}$-in. plate in pipes costs about 25 per cent. more per lb. than 1-in., due to necessity of drilling holes after plate is rolled to cylinder, to avoid making holes oblong. Thin pipe is more expensive than medium-weight, due to larger number of rivets required.

Thickness of plates should be determined as follows:† (1) $t = pdf. \div 2t_sE.$, t = thickness of plate, in in.; p = internal pressure, lb. per sq. in.; f = factor of safety; t_s = tensile strength of plate, lb. per sq. in.‡ = (55,000); E = efficiency of joint; d = internal diameter of pipe, in in. It is good practice to use 15,000 for $t_s \div f$. Constant $\frac{1}{16}$ in. is commonly added to computed thickness as a provision against rusting.

San Fernando siphon of Los Angeles aqueduct is 72 in. in diam. and made up of $\frac{1}{4}$- and $\frac{5}{16}$-in. plate. It was designed for a pressure corresponding to the maximum hydraulic grade line to save expense.[2] The 10-ft. pipe which collapsed due to washouts and sudden emptying, in the Antelope Valley siphon of the same aqueduct, had $\frac{1}{4}$- and $\frac{3}{8}$-in. plate. It was restored to shape by refilling with water. Four miles of the 36-in. Snooke River line were composed of $\frac{5}{16}$-in. plate.[3] In laying, contractors prefer thicker plates, as they make

* Hydraulic Power Committee has prepared specifications for fabrication of riveted steel pipe (1923).

† Specifications, Dept. Water Supply, Gas and Electricity, New York.

‡ Hydraulic Power Committee recommends elastic limit as proper basis for the factor of safety. Water-hammer ignored.

stiffer pipe to handle. A common shop rule is $t = \frac{d}{200}$. Stiffener rings are sometimes provided on penstocks, to avoid distortion in handling. On 9-ft. penstock for Portland Railway Light & Power Co., where plate thickness is $\frac{3}{8}$ in. or less, the pipe is reinforced with outside stiffener rings of $5 \times 3 \times \frac{3}{8}$-in. angles spaced 2 ft. 9 in. on centers, to afford rigidity while handling.[4] For low heads, thickness may be determined by external load conditions, as in 60-in. conduit at Cleveland filters.

Plate Lengths. Specify maximum as well as minimum lap. Edges of plates as delivered from mills are not straight; when riveted, lap is sometimes as much as 2 in. in excess.

Bending Plate. Good practice requires plates rolled cold to true curve, no heating or hammering being allowed.

Beveling Edges of Plates. Plates are often cut and beveled at one operation by a multiple shearing machine. Edges are sometimes beveled by planing. Many persons prefer bevel edges on all plates $\frac{1}{2}$ in. or more thick, to facilitate calking. On the Catskill aqueduct it was required that the bevel be formed at 45° by planing; 60° is sometimes required. If a steel plate is considered laminated, the best calking should be done with bevel edges; planing is preferable to shearing, which is liable to tear the fibers. At least one large manufacturer prefers square edges for calking. Split calking from a square-edged plate is dangerous, as a split may reach some of the rivet holes.

Diameter of Pipe. With alternate inside and outside courses, diameter is measured to inside course. Allow $\frac{3}{16}$ in. in computed diameter for fitting outer course over inner; rivets will take it up when joints are made. With tapered rings, diam. is that of the interior at the small end.

Size of Pipe. Riveted steel pipes 30 in. in diam. are small for good workmanship, although pipes as small as 24 in. have been riveted in the field for short lengths; 36-in. pipe is more easily riveted, and is the ordinary minimum. Pipes have been made up to 18-ft. diam.

LOCK-BAR PIPE*

Lock-bar pipe is in use on 30-in. line, Coolgardie, Australia; on 48-in. line, Portland, Ore.; in 72- and 66-in. Catskill conduits, Brooklyn, in Seattle, and in many other pipe lines. Specification for lock-bars at Seattle:

> Shall be a steel that experience has shown to be adapted to work proposed; shall be equal in quality to steel specified for rivets of riveted steel pipe, and shall be subjected to same tests. It shall stand cold rolling and working.

Lock-bar pipes accepted by Board of Water Supply in 1911, for 66-in. pipes in Brooklyn had plates $\frac{7}{16}$ in. thick. These bars were of "extra dead soft" steel; composition: carbon, not over 0.1 per cent.; sulphur, not over 0.05; phosphorus, not over 0.04; copper, 0.18 to 0.25. This steel proved satisfactory; the pipe maker said it was the best metal yet used in lock-bars. Later the steel mills and pipe makers contended that the copper had been of questionable benefit, if not objectionable. Careful investigation showed no advantage in annealing the bars after rolling, if properly handled on the "hot

* Called "locking-bar pipe" in England; East Jersey Pipe Co. has American rights.

beds" at the rolling mill. Annealing would add a few dollars per ton to the price of bars.

The manufacturers have test data to prove a reduction in plate thickness permissible. The Board of Water Supply, New York, specifies (1923):

Joints formed by welding or special devices, giving strength equal to or greater than that of the riveted joint specified . . . will be allowed on satisfactory proof of efficiency and lasting properties, but no reduction in diameter of pipe or thickness of plate will be considered.

Some municipalities allow smaller diam. for lock-bar pipe; for instance, Hartford[5] took bids on a 42-in. lock-bar pipe, a 42-in. cast-iron pipe, and a 44-in. riveted steel pipe. Cleveland took bids on a 60-in. lock-bar pipe, 60-in. cast-iron pipe, and 63-in. riveted steel pipe for the raw-water conduit in 1923. Maury[54] estimates 10 or 15 per cent. less capacity for riveted steel than for hammer-weld or lock-bar pipe.*

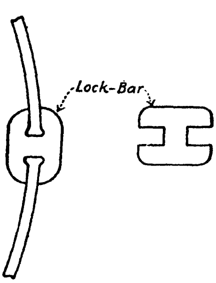


FIG. 137.—Lock-bar pipe. Longitudinal seams.

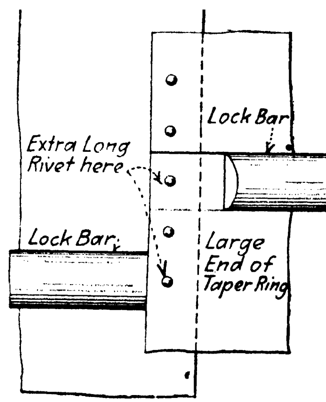


FIG. 138.—Lock-bar pipe. Transverse joints (circular seams).

Fabrication. Longitudinal seams are formed as shown; joints may be figured for 90 per cent. efficiency (tests by Unwin showed 100 per cent.).

Longitudinal edges of plate of lock-bar pipe should be upset to required thickness and after the plates have been rolled to true circles the lock-bars should be closed down over the upset edges by a hydraulic jack exerting a pressure of not less 500 tons. The space between the upset edge of the plate and the bottom of the groove in the bar should not exceed $\frac{3}{16}$ in.; if greater, or if bar shows any sign of cracking, it should be cut out and a new bar substituted. Pipes are ordinarily fabricated in 30-ft. lengths; shorter lengths can be made to fit conditions at curves, valves, etc. Circular seams are made as in riveted pipes. Lock-bars are near the horizontal diam., but stagger on adjacent lengths (see Fig. 138).

Failure. Under a 166-lb. test pressure, a 52-in. lock-bar steel pipe,† $\frac{5}{16}$ in. thick, burst; stress in steel was about 13,800 lb. per sq. in. Lock-bar broke transversely in 11 places, 8 to 15 in. apart, and cracked longitudinally. Metal showed crystallization. Two or three minutes after break, pipe collapsed at three points upstream, first 1700 ft. away. Here nine pipes, each 30 ft.

* See also p. 762.
† At Portland, Ore.

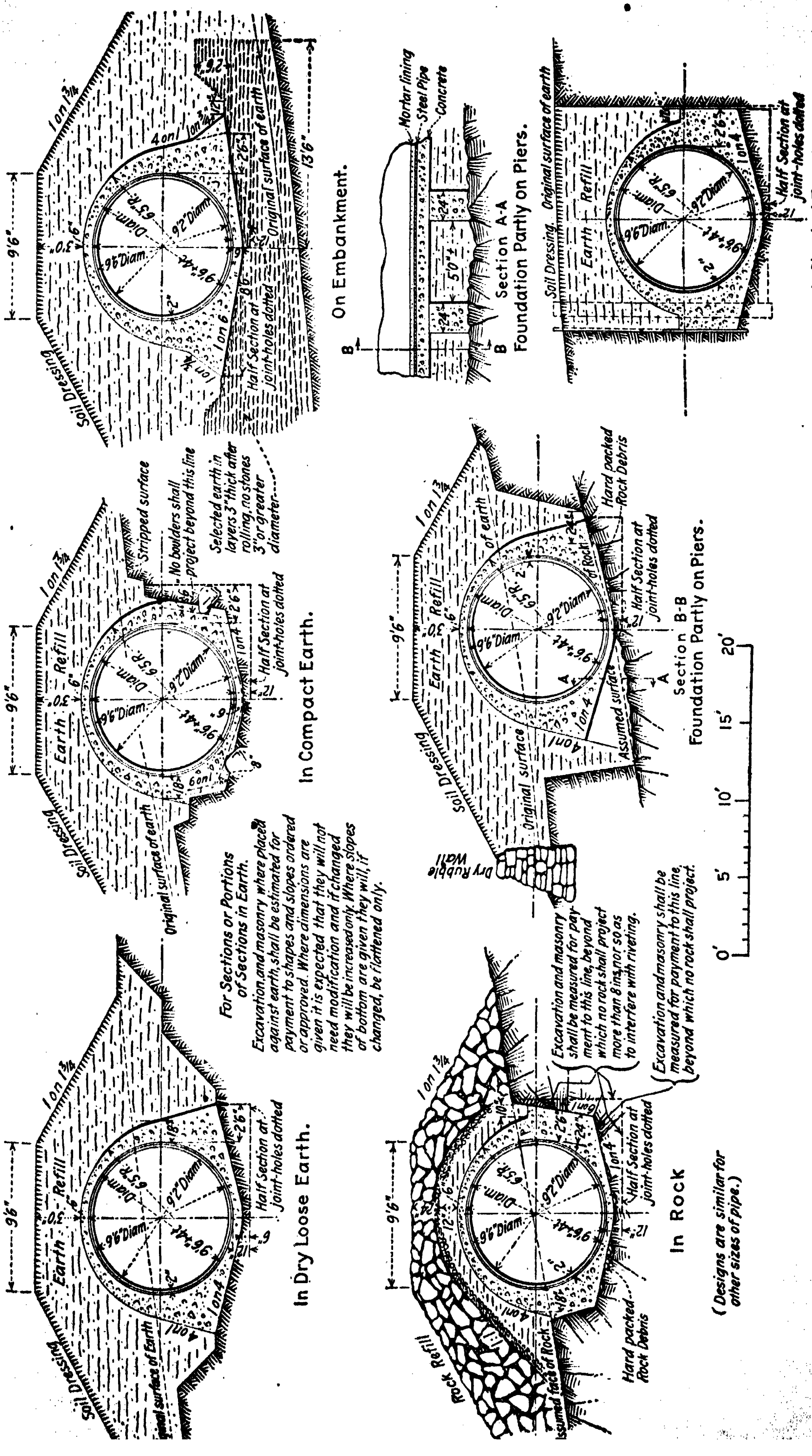


FIG. 139.—Steel pipe siphons, standard types, Catskill aqueduct. Concrete jacket and mortar lining.

long, took this shape: . Pipe was not backfilled; two concrete anchors on either side prevented collapse from spreading. At two points about 1000 ft. farther upstream, nine-pipe and four-pipe lengths, respectively, took this shape at places not backfilled: . There was solid backfilling between breaks. These breaks, due to faulty metal in one bar, are not an argument against lock-bar pipe. Of tens of thousands of bars made in United States, it is stated, breaks could be counted on one's fingers.[6]

PICKLING AND COATING STEEL PIPE

Pickling. Pickling under Catskill aqueduct specification, by East Jersey Pipe Co., was done as follows: Each plate was bent in rolls, which thoroughly broke the scale on the inner side but had less effect outside. Acid and washing tanks of wood, in the same shed with the heating ovens, were set in the ground nearly flush with the floor. Wash water was kept alkaline by occasional addition of soda ash. Acid solution was kept at strength of 5 per cent. of oil of vitriol (latter practically 93 per cent. pure sulphuric acid); it was heated to approximately 125°F. and agitated by steam jet discharging below the surface at one corner. Steel was in the acid tank about 15 min., then immediately lowered into the wash tank, for only 2 or 3 min. After removal from the second tank, pipes, if to be coated, were almost immediately run into the oven and heated for dipping. Pipes, after pickling, were free from scale, and of a uniform steel-gray color. Heat of the acid bath is sufficient to cause the pipes to dry rapidly when lifted into the air from the wash tank. Tests showed that the breaking of the scale by bending or other fabrication helped materially in removing it; 3½ per cent. acid solution was quite as effective as 5. Scale is insoluble in acid, and is removed by the action of the acid on the metal beneath. Weak acid solutions attack cast iron vigorously; a little heat, about 125°, greatly accelerates this action; therefore such fittings should not be attached until after the steel has been pickled, if this be practicable, or cast-steel fittings should be used.

On some of the Catskill aqueduct work, after pickling, whitewash was applied to protect from rust between fabrication and the application of the mortar lining. Heavy lime whitewash was made as follows: To 1 bbl. (about 50 gal.) of whitewash there were added 20 lb. of glue first dissolved in water. Later it was found advisable to add 1 lb. Portland cement to each gal. of whitewash. Brushes proved more satisfactory than spraying machines operated by compressed air. This coating did not prevent light rust, as it did not stay on well enough.

Cleaning and Oiling. In some shops, plates, as received from the rolling mill, are cleaned with dilute acid and brushes before any shop work is done, cleaned again after shop work, and given a coat of boiled linseed oil before exposure to the weather; all painting is then done in the field, preferably with graphite paint. Advantages of oil are that all shop and erection marks made on the bare steel are visible; also any imperfections in the plate which may have passed preliminary inspection may be detected during erection.

Catskill aqueduct specifications for concrete jacket and mortar lining had the following stipulations as to preliminary cleaning: During fabrication into pipe, all plates, at each joint where one plate covers another, shall be entirely

free from mill scale, from any but light rust, and from all dirt, grease, or other foreign matter. Furthermore, when concrete is placed around, or mortar lining placed within, any section of pipe, surface of plate shall be similarly clean. Degree of cleanliness and freedom from scale required shall be such as is produced by thorough sandblasting or thorough pickling, *i.e.*, such that surface of steel itself is exposed. Initial cleaning may be done at the mill, provided the steel is adequately protected and necessary field cleaning is done.

Function of Coating. Coating not only protects the metal from corrosion, but it also improves the hydraulic properties of the pipe. An uncoated penstock of Pacific Gas & Electric Co. developed tubercles and pittings after a few years. Before painting the interior, a sandblast was employed to clean the plates. Two coats of red lead were applied. Aside from prolonging the life of the pipe, this treatment reduced friction losses in the pipe by 25 per cent.[7]

Shop Coating. No shop is equipped to dip pipe over 8 ft. in diam. Dipping vertically in a tank is usually much more effective than painting with brushes. Before dipping, pipes are brought to about 300°, by hot blast or in ovens. Pipes should be revolved in the dipping tank, and great care exercised to avoid foaming, especially while the pipe is being removed; all coating material should be strained through 20-mesh sieve before entering the tank. Some authorities advise avoiding air currents while withdrawing the pipe from the bath; 30-in. Coolgardie lock-bar pipe was placed in a horizontal position and revolved while hardening to give a uniform coating. Current of air blown through it accelerated the cooling. Stewarts & Lloyds, Ltd., Glasgow, apply bituminous lining centrifugally.

Angus Smith Tar Coating. Dr. Angus Smith, in England, about 1840, for protecting iron water pipes, used a coating of coal-tar, distilled to the consistence of melted wax, and an unknown per cent. of raw linseed oil, probably about 5 or 6. After 85 years pipe was found in perfect condition externally, and to contain few tubercles or deposits inside.

Comparison of Coatings. From an examination of various steel-pipe coatings in Eastern United States, it is concluded that a mixture of coal-tar pitch and asphalt has most efficiently prevented or retarded rusting of the interior. Coal-tar pitch is distillate of coal-tar at such a point that naphtha has been entirely removed; the material is then deodorized and reduced to consistence of wax by addition of at least 1 per cent. heavy linseed oil. Ordinary commercial roofing pitch might be suitable after the proper proportions are determined. Some pipe makers state that with modern methods of producing coal-gas and its by-products, coal-tar suitable for coating cannot be had, and that coatings made with available tar do not adhere well; asphalt dips are preferred.

"Coal-tar pitch heated to usual coating temperature, 300 to 400°F., is very thin; it is sometimes used straight, but is more or less brittle and subject to chipping; 'flowing' point is low, 100 to 145°F. Pitches from residues of petroleum distillation vary, Hydrolene being quite soft when applied and yet somewhat brittle and lacking in adhesion to steel (flowing point, 200°F.); Bitose is very soft and full of oil (flowing point, 206°F.); Texaco is harder and tougher (flowing point, 260°F.). Gilsonite, a mineral pitch, is fluxed with some petroleum product having an asphalt base; Sarco is hard, tough and rubbery; flowing point, 260°F. Pioneer has much the same consistence, but a lower flowing point, 245°F. Nubian products are mineral pitch from carbonization of animal fats collected in an earth pocket; corn oil is said to be the flux; hard, tendency to brittleness; flowing points,

230 to 270°F. All are thicker than coal-tar pitch when heated to coating temperatures, and incline to heavier coatings, which do not adhere so closely to the steel.

Tests. "Comparative tendency to soften and flow is shown by placing a pill of each pitch weighing about 0.2 gram near the top of a glass plate inclined 45° in the oven, heated to any desired temperature for 24 hr. Softening and flowing points of the harder varieties are determined upon pieces about 1-mm. diam., placed upon thin microscopic cover glasses laid upon the surface of a mercury bath, the bath and dish being covered with an inverted funnel, through the stem of which passes a thermometer. Temperature is raised 2 to 3°F. per min., particles are examined with magnifying lens and temperature at which the edges begin to round over is regarded as 'softening' point; temperature at which the pitch begins to spread where in contact with a glass is called 'flowing' point. For coal-tar and soft pitches, 1 gram is placed on a washer having an opening 8 mm. in diam., suspended in a 400-c.c. beaker of distilled water about 7 cm. from the bottom; temperature is raised 2°F. per min. and point at which pitch runs through washer and touches bottom of beaker is flowing point.

"For determining tenacity with which coating will adhere to steel: Sample is brought to coating temperature and into it are dipped, part way, clean steel bars about 8 in. long previously heated to same temperature; after about 3 min., the bars are removed, allowed to drain and cool. One set is chilled to 32°F. and another to 0°F., ice and salt being used if convenient; then strike as heavy blow as possible over the edge of an anvil, repeating once or twice on each side, the blow being delivered above the coating. If latter does not chip, blows are repeated upon coated section. Firm, tenacious coating will not chip off at either temperature, except at the point where the blow is delivered; a fair coating may hold at freezing and chip slightly at zero, while a poor coating will be likely to come off in large scales at either temperature.*

"Elasticity may be determined by strips of tin plate 2 in. wide heated to coating temperature and dipped in hot pitch. After setting, the strips should be chilled to same temperatures as the steel bars, and bent quickly 45°. Tough, elastic pitch should not crack.*

Springfield Pipe. "Coal-tar pitch of standard quality mixed with 3 per cent. raw linseed oil was adopted for 60,000 ft. of 42-in. lock-bar pipe, Springfield, Mass., by Hazen & Whipple; applied to 30-ft. lengths by East Jersey Pipe Co., Paterson, N. J. Pitch was run from barrels into a small tank, where it received the linseed oil; it was then drawn into a vertical dipping tank, and kept constantly at 350°F. by steam coils. Pipes were first cleaned of mill scale by wire brushes, a most important step, as rough adhering particles cause spreading of pitch upon draining, resulting in uneven thickness over an area surrounding each particle. Each pipe was then placed on rollers in front of a blast and heated until the end farthest from the blast was sizzling hot; the end nearest the blast was put in the pitch first, compensating somewhat for the slightly lower temperature at the bottom of the tank; top and bottom temperatures were kept within 10 or 12° of each other. Excess coating was drained into the tank and the pipe then stood upright on an iron plate until the coating was thoroughly set and nearly cold. Five pipes were put on each flat car, blocked up free of the floor and protected one from another by canvas strips. With one tank and two hoists, about 1500 ft., or 10 carloads, could be coated in 1 day. Freedom from troubles during coating was remarkable. Constant loss of volatile compounds was compensated by adding tar oil. Fresh pitch and linseed oil were added as the bath was depleted. At one time after the addition of fresh pitch, frothing occurred; bubbles so formed inclined to stick to the pipe and cause uneven coating; probably due to moisture in the pitch; it was

* See foot-note, p. 317.

overcome by the addition of a little linseed oil." * In 1921, 12 years later, the pipe was reported in apparently good condition—it had not been out of service; there had been no opportunity for a thorough examination. In 1926, Lochridge considers the coating successful, although there has been some electrolysis at places where subsequent operations have disturbed the coating.

Protection from Electrolysis. At Springfield, Mass., 42-in. lock-bar steel pipe was protected from electrolysis by covering of two to eight plies of "Barrett Specification" felt weighing 14 to 16 lb. per 100 sq. ft., single thickness, and "Specification" pitch (American coal-tar pitch), 30 lb. each mopping per 100 sq. ft. Felt was cut in strips 12 in. longer than half circumference. Pipe was mopped on top half and felt immediately applied while pitch was hot, strips were laid shingle fashion to break joint every 16 in., for as many plies as required. After covering top half, pipe was turned over and lower half was treated in same way. Where pipe was subjected to direct currents, eight plies were used; as distance from current increased, number of plies were lessened. After pipes were riveted in trench, joints were similarly wrapped.[8]

Bitumastic enamel is a refined coal-tar composition with 20 per cent. or more of mineral filler to give the proper body or consistence. From 40 to 60 per cent. of the coating is soluble in carbon disulphide and the melting point under the ball-and-ring test is from 160 to 170°F. The bitumastic solution is of the same composition without the filler; it has been cut with a naphtha solvent so that it is liquid at all temperatures. Both the Hermastic of Wailes-Dove-Hermiston Corp. and Briggs Tenax solution and Briggs ferroid enamel† were used on the recent Brooklyn conduits.

Bitumastic enamel should be applied only by experienced men, as the material can be easily ruined. Material can be heated too much, making the coating hard and brittle, but increasing the covering capacity. Thinners will also ruin the material while increasing the covering capacity. If applied to a damp, greasy, or dirty surface, the enamel will be thrown off the steel pipe in a short time. Applying the enamel over a solution of a different base will produce poor results. Conduits installed in 1913 under a specification requiring the bitumastic enamel brushed on thin to get a smooth surface ($\frac{1}{16}$ in. was maximum thickness allowed) were inspected after 7 years; the enamel was found in excellent condition, and better than other experimental coatings applied simultaneously. The thinness of the coating was largely responsible for a few small tubercles on the invert where some walking took place after the line was finished. Recent specifications have called for a thicker coating.[9]

Graphite (Rochester specifications, 1916).[10] After cleaning in the shop, one coat of boiled linseed oil shall be painted on by hand. After pipes are delivered at site, they shall be painted by hand, both inside and out, with Detroit graphite paint No. 501 and, after a sufficient period for hardening of this coat, it shall be followed in a similar manner by one coat of Detroit Graphite Paint No. 106. Both paints shall be received in original packages and shall be thoroughly agitated

* From "Pipe Coating," by G. C. Whipple, before Chemists' Club, Polytechnic Institute, Brooklyn, 1909. There have been some improvements since 1909 in determining melting point. See also "Methods for Testing Coal Tar and Refined Tars, Oils and Pitches Derived Therefrom," by S. R. Church, *J. I. E. C.*, April, 1911; and "Tars, Old and New," by same author, *J. N. E. W. W. A.*, December, 1922.

† Briggs Bituminous Composition Co.

with a paddle before being dipped from the original containers. The paint as delivered shall be used without the admixture of any oil drier or thinning material. During coating and period of drying or hardening, pipes shall be kept dry and under cover from weather. When both coats are sufficiently hard, or approximately a week after second painting, pipes shall be painted by hand with one coat of Trus-Con special paint.

Roofing paper has been applied to 4- and 8-in. pipe in California (the former having welded joints, the latter, screwed joints), as a protection from corrosive adobe. Two coats of California asphaltic paint were first applied, after which the pipe was covered with high-grade roofing paper, held with wire.

Coolgardie 30-in. lock-bar pipe* was heated to 300°F. and immersed in a mixture of Trinidad asphalt and tar, maintained at boiling point (14 per cent. of 350-mi. length was coated otherwise). Before the coating was entirely hard, find sand was distributed over the outside and slightly pressed in by rollers; this was expected to retard flowing under high sun temperatures. Australian temperatures range from below freezing to 170° F.† Chief Engineer Reynoldson thinks sanding not desirable. Soft coatings, with a tendency to flow, are not as reliable as hard coatings. Hard coatings may peel from brittleness in cold weather, but are denser, more durable, and in closer contact with the metal.[11]

Burlap covering saturated with hot asphalt compound has been spirally wound on to lock-bar and riveted steel pipes by East Jersey Pipe Co., New York, at moderate cost. Small seamless pipes wound spirally with strips of the burlap saturated with asphalt can be obtained in the market. There are conduits so treated at Winnipeg, Minneapolis, Montreal, Rutland, Vt., and Brooklyn. The last mentioned was laid by Board of Water Supply in 1915.

The 8-in. wrought-iron pipe for oil delivery in the Tuxpam Roadstead, Mex., was delivered in 20-ft. lengths and screwed up on shore, with couplings, 7⅞ in. long. Before launching, the pipe was coated with "Texaco Dip," applied hot, and was wrapped with burlap 20 in. wide.[12]

A 4-in. high-pressure gas line in California was wrapped in the field with a prepared roofing fabric of burlap, asphalt, and felt, which came in rolls. Strips 20 ft. long by 16 in. wide were prepared; this width provided a 2½-in. lap, as the pipe circumference was 13½ in. A heavy pitch heated to 150° F. was mopped on the metal, and on the side of the fabric which would be in contact with the metal. Fabric was immediately put in place and smoothed down promptly to close contact. No. 18 wire was then wound spirally and the whole surface given a final coat of hot pitch. "When cool, this presents a hard, waterproof surface that cannot be penetrated."[13]

Spring Valley Water Co., San Francisco, Cal. Directions for Asphaltum Coating. Santa Barbara asphaltum free from impurities broken into 2- or 3-in. pieces. Place in a melting trough and fill the interstices with coal-tar free from oily substances. Boil slowly 4 to 7 hr. stirring frequently and allowing the refuse to collect in the bottom of the trough. Proper temperature to insure toughness and tenacity of dip is 300 to 305°F. After boiling, the

* See also p. 325; also Longley in *J. N. E. W. W. A.*, Vol. 39, 1925, p. 421.
† Black pipe in sun.

refined asphaltum is poured into another (the dipping) trough, and the refuse is taken from the boiling trough. During the process of dipping pipes, the dip is kept at the same temperature. Consistence is maintained by addition of refined asphaltum from the melting trough, and by the occasional addition of coal-tar, to prevent the consistence in the dipping trough from becoming too stiff. Pipes should be given two dips, the first lasting from about 2 to 20 min., or until pipe has attained temperature of bath. Pipe is then taken out and drained into dipping trough. After it is thoroughly drained, it is immersed again to obtain thickness. It is then taken out and drained into dipping trough. After it is thoroughly drained, it is again immersed for 3 to 5 min., hoisted out, and drained again. In a long run of mixing, the proportion of tar to asphaltum was found to be $\frac{2}{3}$ bbl. (50 gal.) of coal-tar (weighing about 420 lb.) to a short ton of asphaltum. In a case in mind, the melting trough contained a little over 50 tons, and in the preparation of the dip (tempering) 44 bbl. of coal-tar were used. This shows about 1 bbl. coal-tar to 1 ton asphaltum. Amount of material necessary to purchase will exceed material used by 10 to 15 per cent. Material required per sq. ft. of surface inside and outside (for two coats) runs from 0.8 lb. for small pipes, of thin iron, to 1.15 lb. for large pipes, of thick iron. These specifications have been used since 1868. A 30-in. pipe (No. 11 gage) so coated has been in service since 1868 under 60 lb. pressure.*

Water-gas and Coal-gas Tar Paint,† U. S. Reclamation Service.‡ Specifications for metal work: All metal work shall be thoroughly cleaned of all loose scale and given one coat of water-gas tar followed by two coats of coal-gas tar. All coats shall be applied when the temperature of the air and metal is not less than 60°F. Water-gas tar shall have a specific gravity not less than 1.05 nor more than 1.10 at 60°F. and shall be of such consistence that it can be applied with brushes. Melting point of coal-gas tar, as determined by cube method, shall be between 105 and 110°F. Both water-gas tar and coal-gas tar shall be freed from moisture, and all fats shall be extracted from water-gas tar. If water-gas tar is too thick to spread, contractor may use suitable light oil for thinning. Water-gas tar may be applied without heating, but first coat of coal-gas tar, which may be applied a few hours after application of water-gas tar, shall be applied hot and brushed out thin as possible so that the coating will not run nor peel after it is dry. Second coat of coal-gas tar shall not be applied until after the first coat has set. Contractor may use water-gas tar without the fats extracted, in which case first coat of coal-gas tar shall not be applied until water-gas tar has set, usually at least 4 days. This tar paint shall be well worked into all joints and open spaces. Machine-finished surfaces shall not be painted, but coated with white lead and tallow.

This paint is used for underwater metal work and wood-stave pipe, and in flumes to prevent erosion as well as corrosion; for the latter it is the only satisfactory paint. It is superior to any other for all submerged metal work and believed to be good for any place where a black paint with a slightly sticky surface is not objectionable. In winter of 1912–1913, penstocks and draft tubes in Minidoka power plant, except No. 1, were painted with red lead, carefully compounded and applied in accordance with recommendation of one of largest paint manufacturers. No. 1 was painted with water-gas tar and coal-gas tar. At end of 1913,

* Information from G. M. Elliott, Chief Engineer, 1922; see also Leonard Metcalf, *E. C.*, Dec. 30, 1914.

† Information from R. W. Walter, Assistant Chief Engineer, by courtesy of Chief Engineer F. E. Weymouth.

‡ See also p. 80.

red lead showed decided deterioration. Tar paint was in very good condition. At end of 1914, red lead had almost entirely disappeared where it came in contact with water. Tar paint was in good condition in penstocks, but was getting thin on outsides of draft tubes. Tar adhered well and when scraped off left metal bright. Conduits 4 ft. 4 in. in diam. through concrete Arrowrock dam were very carefully made with wooden forms, surfaces trimmed, scraped, washed with grout, and then painted with two coats of water-gas tar and two coats of coal-gas tar. Water-gas tar was used thin, so that it soaked well into pores of concrete, causing coal-gas tar to enter concrete and bind particles together, completely filling voids at and near surface, and giving concrete a slick and fairly durable finish cheaply and easily applied. After a season's service under velocities up to 64 ft., this paint was in good condition. In fall 1915 a metal flume on Sun River project was painted with various brands of paint. The tar paint did not conform perfectly to specifications (fats were not removed from water-gas tar; coal-gas tar was too thick and only one coat was applied). In December, 1919, after four seasons' service beneath water, one proprietary paint had completely disappeared and others were peeling badly; tar paint was peeling in places and metal rusting beneath. The paint on the outside was still intact, however, and also on the inside above the water line. In 1914, a flume was installed on Boisé project to test rust-resisting qualities of various brands of metal sheets, both black and galvanized, and incidentally to determine value of tar as protective coating. Tar paint stood the test of 4 years under water excellently. Metal was new when paint was applied and there was a possibility that, on account of smoothness, it would peel, but there were no signs whatever of peeling, and, especially on the galvanized sheets, it looked almost as good as when first applied. On Uncompahgre project a number of flumes were given a coat of water-gas tar followed by a coat of coal-gas tar, thinned with water-gas tar. In a number of cases the coating was too thin, either running to bottom of flume or checking and cracking on sides exposed to sun. Since summer of 1918 coal-tar has been used without thinning, with good results. Best results have been obtained when tar was applied before flume had been used more than one season, and when tar was applied in warm weather. When applied in cold weather, it would not stick, but blistered and peeled. It was found essential to clean it thoroughly, removing all rust from sheets and joints, and to have them perfectly dry and free from dust. Tar was applied hot with heavy brushes or burlap swabs. Joints should be retarred every fall and entire flume every second fall. A heavy coating of tar has proved best protection for flumes.

Protective value of tars and method of applying them are not well understood by trade and by structural manufacturers. Producing companies are primarily interested in gas and usually make no attempt to control tar. Tar varies according to coals or oils used and conditions incident to manufacture of gas; products from one locality may differ from those obtained from another. Not all tars are suitable for paint; all materials should be purchased under definite specifications. It is not always possible, however, to require a given plant to meet requirements. The value of tar sold for paint is too small in comparison to value of gas to justify any change in materials or methods. It is a case of selection rather than of control. A number of plants have paint departments which specialize in refinement of tar for protective coatings. Water-gas tar as it comes from the retort contains a large quantity of water and fatty oils. So-called refined tar has only the water removed. The oils render the tar so slow drying that it is inconvenient for use as a paint. When fatty oils, which are claimed to add nothing to value as a protective coating, are removed, the product, water-gas pitch, will not flow, but can be cut or thinned to consistence of paint by light oils or creosote, or light petroleum distillate. From 40 to 60 per cent. of thinner is usually required. It is claimed that

certain acids in creosote tend to react with steel; hence creosote should not be used in paint for metal work. On account of its preservative properties, creosote is desirable for use on wood. Light oil or gasoline, when used for thinning, entirely disappears as coating dries and serves no useful purpose other than to assist in spreading the pitch. Coal-gas tar can be obtained of any consistence, from that which can be applied with brushes without heating to that which requires considerable heating. Tar having a melting point of 105 to 110°F. by cube method is plastic at 50°F. but not liquid, nor hard and brittle like pitch. It may be thinned by heating or by adding creosote or light oil; for brush work or dipping heating is preferable. If paint is to be applied with spraying apparatus, the tar cannot be heated, since mixture of air and hot tar is explosive. Both creosote and light oil used for thinning tar paints are distilled from coal tar; the light oil has the appearance of water. Tar paint is economical in first cost, but must be worked in an entirely different way from ordinary paint and many painters find it difficult to handle at first. No painting should be done on chilled or damp metal. If nature of articles permits, they may be dipped. A quantity of light steel pipe was so treated on Okanogan project, Washington, and results appear to be satisfactory.

Quantity of tar paint required to cover a given surface depends upon consistence, method of application, and nature of surface. Roughly, 1 gal. of water-gas tar will cover about 160 sq. ft., and 1 gal. of coal-gas tar about 80 sq. ft., for one coat each.

Los Angeles waterworks has used nothing else for 13 years. Some aqueduct siphon pipes were repainted outside only after 10 and 11 years' service. They were very durable where exposed to elements and buried parts are found on uncovering to be as bright as when laid. Inside these pipes the coatings seem to be almost entirely intact. Chief Engineer Mulholland strongly recommends use of water-gas tar for painting steel pipe.

Tar-cement Paint. A paint made by mixing 20 per cent. of dry Portland cement with any good grade of coal tar, heating, and then thinning with a small quantity of kerosene, if necessary, is recommended by American Rolling Mills Co. for smokestacks, culverts, pipes, and other iron and steel. Ordinary coal-tar contains some acid, which the Portland cement neutralizes. It is used by flume manufacturers. In Southwestern United States, where alkaline conditions are severe, tar-cement paint is recommended for culverts and flumes by Federal Bureau of Public Roads.

Coating at Field Joints. It is not necessary to remove coating, except where it interferes with putting pipes together; it is melted by hot rivets and helps to make joint tight; it may be claimed that coating prevents sheets from coming into actual contact and therefore weakens joints by acting as a lubricant, but trouble on this account is not experienced in practice.

Coating Outside of Pipe Only. Asphalt or a similar dip coating may be confined to the outside of the pipe by whitewashing the interior and then scraping off the coating after the pipe has been dipped. This is the method used in keeping machined flanges free from coating, and would cost at least twice as much as coating both inside and outside. To close the ends of the pipe with bulkheads and load it so as to sink into the dip is less practicable.

Repairs to Coating. If necessary to patch scarred places, one method is to heat the coating by a plumber's blowtorch until the old coating all around is fluid; when the defect is small, the old coating can be made to flow over it, but if large, some coating material previously heated is applied with a brush

and the flame turned on, making the new join the old. Coating will take fire if the torch is not used carefully; it is hard to extinguish, especially in summer.

Defects in Coating. A perfect, homogeneous coat would be a non-conductor. Coats generally contain minute air or gas bubbles, and abrasions occur during handling and transportation; also the coating itself may not be impermeable under high pressure; thus water finds direct contact with the metal and electrolysis starts. No coating has proved permanently successful; in a short time elasticity and adhesion are lost.

Observations show that, at first, coatings usually adhere on nearly the entire surface. Repeated examinations of pipes in service have shown that most, if not all, bituminous coatings deteriorate greatly in a relatively few years. Asphalt coatings seem to lose elasticity and become brittle. After 2 years' service, coatings, particularly those that go on heavily and lack tenacity, form blisters, ranging in size from those the size of a pinhead to those the size of a hen's egg, holding alkaline water which contains more solids in solution than the water passing through the pipe, but the iron or steel beneath will generally be bright, and tubercles will be found in the vicinity. Sometimes pittings occur under the tubercles. They have been found 0.093 in. deep after 10 years' service. Danger is that a blistered coating will eventually expose a large surface of pipe. Blisters can be produced quickly by passing a current of low voltage through sea water, using strips of sheet iron for positive electrodes and similar coated strips for negative electrodes, with edges protected by paraffin; 12 to 24 hr. will produce blisters with some coatings. The walls of large blisters are about $\frac{1}{50}$ in. thick; they are easily broken. Blisters generally are largest on the lower quarter of a pipe and decrease in size and number toward the top. Experience in Eastern United States has been that steel pipes coated with any asphalt compound show a decided reduction in carrying capacity after a few years.* This loss results from: (1) growth of vegetable matter; (2) deposit of mineral matter; (3) formation of tubercles of rust;† (4) formation of blisters in coating. As to (3) and (4), experience on some parts of the Pacific Coast has been different, pipes of considerable age having developed neither.

Asphalt coating of a 42 in. pipe in Paterson, inspected when 20 yrs. old, had cracked and shrunk until but 33 per cent. of the original coating was left An 8-inch pipe, of same age and coating, had coating nearly intact. Possibly the diameter has a bearing on the tenacity of the coating.

Examination of a large number of preparations, applying tests, showed that asphalt preparations and petroleum products do not adhere closely to steel; petroleum products are most resistant to the action of water, but no more so than coal-tar, which has the advantage of being tenacious with less tendency to blister, especially when used with about 3 per cent. of linseed oil.‡

MORTAR LINING AND CONCRETE JACKET

Mortar-lining experiments§ made for the Catskill aqueduct showed that a lining $1\frac{1}{4}$ to $2\frac{3}{4}$ in. thick of mortar or mortar and tile is of sufficient strength

* The same is true of old cast-iron pipes; see p. 387.

† These were removed by sandblast on a penstock of Pacific Gas & Electric Co., and friction head of newly painted pipe was reduced 25 per cent.[14]

‡ At Lawrence, Mass., are riveted wrought-iron pipes, some coated with coal-tar pitch, and some with red lead, in good condition after 40 years.

§ Described in 1908 Report, Board of Water Supply, p. 68.

to maintain its position within a 9-ft. or larger pipe, acting as an arch; in smaller pipes the arch would be correspondingly stronger. It is feasible by several methods—grouting around a cylindrical form or a mandrel preferred—to produce a hard, smooth lining in a pipe. Durability of steel protected by concrete or mortar depends not so much on the ability of the lining to exclude all water as to stop circulation in contact with the steel.

In the Catskill aqueduct 14 minor valleys are crossed by three steel pipes, of which the first was laid in 1909, and the second and third lines in 1921–1923, thereby securing the benefits of deferred investment. Total length is 33,031 ft.; heads range from 50 to 340 ft.; diam., 9 ft. 6 in. to 11 ft. 3 in., in original pipes; smaller diam. in some later pipes after hydraulic tests had established capacity of 1909 installation. Plate thickness varies from $\frac{7}{16}$ to $\frac{9}{16}$ in. The following description applies to the earlier pipe, although mainly true for later pipes.* In view of knowledge concerning the usual coatings, the engineers decided to clean the plates, cover the pipes, after laying, with a 6-in. jacket of concrete, and line with 2 in. of cement mortar. Pipes were tested at the shop and shipped in 15-ft. lengths, laid on concrete cradles in the bottom of the trench (Fig. 139), tested in trench under normal hydrostatic pressure and made tight, and, while still full of water and under full pressure, jacketed with concrete. These precautions were necessary, as variation in shape would crack the jacket; pressure was maintained until the jacket had hardened, the water was emptied out, the pipe cleaned, and the lining put in. The lining in these Catskill aqueduct pipes consisted of 1 part Portland cement and 2 sand, approximately. The first step in lining was to place the invert about 8 ft wide, screeded to template; cylindrical wood forms in segments were then set up and the remainder of the lining poured, as a thin grout or mortar, through holes in the top of the pipe. These holes were tapped for 2-in. pipe in the uphill end of each 15-ft. length, through which hot rivets were passed, and later mortar poured. Mortar, or grout, was mixed to a thick, creamy consistence and allowed to flow into place as uniformly as possible. When the section was filled to the top, pouring of grout continued until grout ran from the air inlet; then headers of steel pipe were screwed into the inlet and outlet and filled with grout so as to put a head of at least 4 ft. on the highest part of the section; the headers were kept filled with grout until the grout had set, the pipes were then removed, and the holes made watertight by screw plugs. Some fine cracks appeared in all linings during the first winter, pipes being empty; this was not a cause for apprehension. One siphon was lined with a cement gun; this lining was built up in successive layers and finished with a trowel to a very smooth surface. After 15 years, linings are in excellent condition. It is important not to let the lining dry quickly.

Adhesion of mortar lining and concrete jacket was not absolute everywhere, as was proved by sounding, but the separation was very slight, as was determined by cutting into selected hollow-sounding places. Some cracks and separations had been predicted and probable results investigated at the laboratory before the contracts were prepared. In one test six steel plates 8 × 16 in., 12 gage, were pickled, then rubbed with emery cloth and placed

* See *J N E. W. W A*, Vol 25, 1911, p 345, *E N R*, May 17, 1923, p 866, and also Sanborn and Zipser, *T. A. S. C E*, Vol 83, 1920, p. 1052.

horizontally in a tank, separated from the bottom of the tank by alberene stone blocks and from each other by wood strips. First pair was without protective covering; second pair had upper surfaces protected by 2½-in. mortar slabs separated from the plates by 0.04-in. metal strips; third pair was protected by 2-in. mortar slabs cast directly on the steel and apparently adhering firmly. Tank was filled with Croton water 4 in. above the top slab and renewed twice monthly. After 2 years the first pair showed heavy corrosion; second, very slight corrosion, most of which washed off; third, part of surface was clean and wet, the remainder dry and covered with firmly adhering particles of mortar, no rust being found when the mortar was broken off. In another test four concrete slabs 15¼ in. in diam., 3 in. thick, were made of a rather dry mix, 1 part of cement, 2.7 gneiss screenings, 6.3 crushed gneiss, by weight; four soft steel rods, ¾ in. in diam., 3 in. apart, were placed in the middle of each slab, so that there were at least 1½ in. of concrete in all directions around each rod. When set, two slabs were immersed in water in tanks 2 ft. deep; two had bottomless galvanized-iron cylinders, 15¼ in. in diam., cemented to them and water maintained 20 in. deep. Latter slabs, submitted to percolation, leaked freely at first, but became gradually tighter; during the last few months there was little leakage. Tests were made in open air, July, 1907 to March, 1909—20 months. Slabs were broken and all rods found perfectly free from corrosion. Concrete as broken was found thoroughly saturated, showing that the water had full access to the rods. Experiments briefly mentioned in *E. R.*, May 8, 1909, indicate that painted bars embedded in concrete and subjected to moisture rust more quickly than unpainted ones.

Merits of Mortar Lining. The carrying capacity is increased 25 per cent. over that in unlined new steel pipe of same waterway.[25] The lining affords protection for the metal. In Little Falls, N. J., a 66-in. steel filter influent pipe had ¼-in. mortar lining (1 Portland cement: 2 sand) plastered over several coats of neat Portland cement grout on an asphalt coating. After 5 years' service it exhibited no cracks nor tendency to loosen, and no rust had appeared, neither was the elasticity of the coating lost, although sections of the pipe not so treated had no longer any elasticity in the asphalt coating and rusting had extended right up to the mortar.

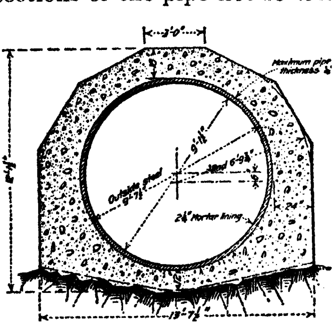


FIG. 140.—Red Mountain Bar siphon, Hetch Hetchy aqueduct.[19]

Red Mountain bar siphon, Hetch-Hetchy aqueduct,[16] consists of riveted pipe, 9 ft. 6 in. in diam. fabricated in shop in 24-ft. lengths, each of three 8-ft. courses; plate thickness, $\frac{9}{16}$ to ¾ in.; weight of field section, 12 to 16 tons; metal protected (Fig. 140) by a jacket of 1:2.5:5 concrete from 18 to 24 in. thick, and a mortar lining 2¼ in. thick, of 1 cement: 1.5 sand. Forms for lining were sheathed with staves furnished by wood-pipe makers. Mortar was poured through 2½- in. saddles at 16-ft. intervals.

Other Projects. At Newton, Mass.,[17] 80-in. steel pipe was lined with 2 in. of Portland cement mortar, for Metropolitan Waterworks. For Jersey

City conduit, 72-in. steel mains under Hackensack River were concrete jacketed; see *E. N.*, Mar. 12, 1903. The recent line has 3-in. shell reinforced with mesh, placed by cement guns; see *E. N. R.*, July 24, 1919. A 24-in. line in California has a reinforced-concrete casing designed to resist full head when the steel pipe rusts away.[18] A 33-in. steel pipe of Dominguez Water Co.,[58] corroding in alkaline soil, was reclaimed by jacketing with a 2-in. shell of concrete reinforced with heavy wire.*

DURABILITY†

Coolgardie pipe,[20] as laid, was but ¼ in. thick for heads up to 390 ft. Annual cost of maintenance varied from $107 to $707 per mile (average $257) for the first 12 years (1902–1915). The original cost was $10,700,000.‡ Examination after 12 years showed original coating good where pipes lie above ground, but more or less perished below ground. External corrosion took three forms: rusting, pitting, and scaling; about 4500 holes in 12 years, or one every 400 ft., are due to external corrosion. Scaling occurs around leaky joints in salt soil (morrel gum); it is hard to detect and is, therefore, the most dangerous form of corrosion. Some pittings were ⅛ in. deep. Gore[21] claims unfavorable factors: exposed in transit, and left lying in hot sun as long as 2 years before laying. The carrying capacity was reduced by 44 per cent. by the nodules formed. See Longley, *J. N. E. W. W. A.*, Vol. 39, 1925, p. 421.

Deterioration of Steel Pipe in Akron.[22] Thirty-six inch lock-bar pipe, laid 1913-1914, 11 mi. long. After 5 years a stretch 1 mi. long in wet clay evidenced severe corrosion. Mild stray current was found flowing on pipe and leaving it in the corroding area 3 mi. from nearest trolley tracks, to follow a route of low resistance in the natural ground back to the equally distant power house. Construction was thoroughly inspected. Coating was specified as hot "coal-tar pitch varnish," but had to be altered to "Pioneer Mineral Rubber," owing to difficulties in controlling mixture and temperature. Construction complications left the pipes exposed on ground throughout one winter. Mineral-rubber coating broke down where it had been in contact with the sod. Final inspection revealed loosening of interior coating at field joints, necessitating repairs. Pipe was put in service August, 1915. In November 1919, leakage disclosed two holes; pipe was found badly pitted outside with numerous blisters, from ⅛- to 1½-in. diam., inside; 20 holes developed to 1922. Soil conditions (sand, clay, shale, ashes; in general, wet) are termed by Professor Veazey "favorable to rapid corrosion of steel pipe." Pyrite-bearing shales and clay are especially destructive. Bituminous coating is a detriment if it becomes porous and spongy. Wet conditions, rather than electrolysis, are held the cause of this pipe failure. Constituents of the soil and its moistness (causing local electrolysis), rather than stray current, are probable cause of corrosion.

Penstock Deterioration.[23] Where a porous backfill allows free drainage away from painted steel but slight deterioration has been found in 20 years. Impervious clays have proved injurious; pittings ⅛ in. deep have resulted. In all soils where the backfill extends only to the horizontal diam. of the pipe, corrosion seems most rapid on the sides and for a depth of 12 in. below the

* Length, 350 miles.
† See also Chap. 36, p. 811.
‡ Steel pipes centrifugally lined with concrete are being promoted by East Jersey Pipe Co., M. W. Kellogg.

ground line, due, apparently, to extreme variations of temperature and moisture.*

Other Experiences. Pittings in 17 year old New Bedford steel pipe attained maximum depth of 0.11 in., or one-third the pipe thickness.[24] At Pasadena,[55] distribution pipes, 4- to 30-in. diam., have been in service 40 years. For Rochester experiences, see report by John F. Skinner, 1913, on "Steel Plate Pipe II." Steel pipes laid across tidal meadows for Atlantic City, Bayonne, Jersey City, deteriorated extensively. Steel pipe siphons (diam., 7 ft. 6 in.; plate, $\frac{7}{16}$ in. thick) at Sudbury River and Happy Hollow on Weston aqueduct after 17 years' service (1923) showed pittings on interior up to 0.15 in. thick—34 per cent. of initial thickness. Pittings are 0.10 in. deeper than when inspected in 1908. Pipe arch bridge, built 1900 over Sudbury River (90-ft. span), has not leaked nor shown signs of distress.

Life. For Catskill aqueduct studies 35 years was assumed as the renewal interval for steel pipe with the best dip coatings; so far as known, no steel main has been in use for that period. Bids for steel *vs.* cast iron for 60-in. raw-water main for Cleveland filters were compared on the basis of 40-year life, although the engineers of the Water Department "feel that 50 years will be more nearly the case." Life of steel pipe with the usual coatings may be assumed 25 to 40 years. Steel pipes at Newark, N. J., about the oldest in Eastern United States, were laid in 1890–1891 and were in service in 1926. Sections of Newark conduit, removed by Foulks in 1925, indicated a further life of 15 to 30 yrs. Electrolytic action may destroy steel pipe in much less than 35 years. Electrolysis of St. Louis lock-bar pipe is described in *E. N. R.*, May 6, 1920, p. 911. See also reports of Am. Comm. on Electrolysis.

LAYING RIVETED PIPE†

Placing Pipe. Calking edges of longitudinal seams should be set face up. Lay tapered pipe with the large end upstream. Put no permanent blocking under the pipe; block up for riveting and remove the blocks as backfilling progresses. Artificial ventilation for inspection of the inside of the pipe is a necessity under some conditions. After the foreman says a length of trench is ready for pipe laying, it should be carefully inspected to see that it is down to grade and that the sides conform to required dimensions; for the latter, a template is useful. Any projections inside the limits should be removed before the pipe is lowered into the trench. While the pipe is on skids over the trench, it should be carefully inspected inside and outside to see that the coating is intact; injuries should be repaired. In entering a pipe into one already laid, great care should be taken to prevent distorting edges of plates. In temporarily bolting a new section to that already laid, the top of the suspended pipe is entered into the other and a bolt inserted through the top rivet hole. Care should be taken to see that longitudinal seams are all spaced approximately equally on each side of the center line. Pipe is then gradually lowered and bolts inserted on each side of its top. To guide the inside plates, pries or bars with sharp, chisel-like ends are used. In case the edges of either plate are distorted, they should be hammered just enough to permit entering.

* See also "Corrosion of Metal Conduits," by Hugo Kühl, *Gas u. Wasserfach*, Vol. 65, 1922, pp. 99–102, for a review of literature.

† Applies also to lock-bar pipe, except as to longitudinal seams.

While the pipe is being adjusted to line and grade it should, if practicable, be suspended from the top of the trench by a sling and braced on each side. Should rivet holes come "blind" more than $\frac{1}{16}$ in., remedy this before leaving the pipe. For example, when ready for bolting, pipe might lie in position first shown (Fig. 141). To bring the pipe to line bolt a few holes on side *a* and then jack end *e*. Thus the rivet holes will come fair at *a* and will pass each other at *cb*; when the holes are cut out for the rivets, the plates will cut back from the lap; thus not weakening the joint. When the holes come blind, so that slight drifting will not remedy them, the joint should be fitted before the rivet gang starts on the seam. In Fig. 141 the part *mnop* should be gouged out, reamed, or drilled, as determined by the inspector. On no condition except when rivets are driven from inside of pipe (as in the bottom), should the part *efgh* be cut out, for if it is the rivet head will not cover the hole. It may become necessary to drill an entire new set of holes; the inner course should be pushed well into the outer course and drilled from inside. All burrs should be removed from holes by a countersink or burr reamer. Circular seams should be well bolted before leaving, using bolts of same diameter as the rivets, spaced not exceeding six holes apart. In no case should drift pins be allowed to remain in holes in place of bolts.

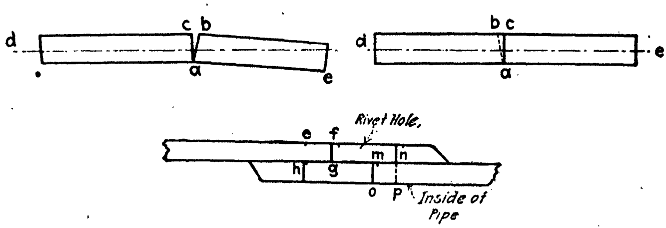


FIG. 141.

In constructing a steel-pipe siphon across a valley with steep sides, movements due to temperature may be nearly avoided by laying the pipe upwards both ways from the valley bottom. On recent siphons of the Catskill aqueduct where pipes were laid downhill movements were experienced—15 in. in one case, and in another case a length of 540 ft. elongated 8 in. down a 19-deg. slope.[25]

Field Riveting. As many rivets as possible should be driven from outside, using a snap to form the heads. These can be driven to the points where radial lines make an angle of 45° from the bottom with the vertical. Two methods may be employed: either to use a snap, or to make a hammered head, as in boiler riveting. Riveters, especially boiler riveters, prefer a hammered head, as they say it insures greater tightness. This may or may not be true. It is true in case holes come blind and have to be cut out, as then a hammered head entirely covers the hole, whereas a snap head may not. It is easier to make a hammered head than a snap head, as a snap has to be struck with a heavy sledge and room inside the pipe is necessarily limited. All seams driven on the bank should have snap heads, as the joint is accessible all around by rolling the pipe over. Plates should lie in close contact at both calking edges. If not well laid up, sufficient rivets should be cut out, the plates hammered up,

and new rivets driven. After a seam is driven, all rivets should be carefully inspected. Any rivets with defective heads should be cut out. All rivets should be tapped with a light hammer, at same time holding a finger against the rivet head. A loose rivet is thereby at once detected. Loose rivets and those that jar *at all* should be *cut out* and new rivets driven. *Do not* allow a loose rivet to be calked. When rivets are cut out, it is likely that rivets close by may be loosened, and if any are found these should be cut out also. Sometimes the head of the last rivet in a longitudinal seam may lap over the edge of the plate. This should not be allowed, as the head will not lie in close contact and a leaky rivet might result; the plate should be chipped before driving the rivet. Or the plate may lap over a rivet head; this is worse, as poor calking would follow. Chip the plate here, too, so that it will lie in close contact with the plate in the other course. When holes come so blind that hammered heads will not cover the holes entirely, burrs should be driven into that part of the hole not filled by the shank of the rivet, and a perfectly tight job will result.

Field Calking. Wherever possible, field calking should be done by machine. Before striking with a calking tool, the plates should lie in close contact at the beveled edge. The seam is first gone over with round-nosed fuller tool; after this a half-round fuller tool is used to finish off. To make a fine appearance a cold chisel may then be used to cut off the thin scale of metal made by the fuller tool. This is not necessary, however, as the seam will be tight after using the half-round tool. Split calking, which is splitting the edge of a plate so as to make a beveled edge lie in close contact with another plate, is not allowed under some specifications. On the Sooke Lake 36-in. conduit, split calking with an air hammer produced the desired results.[26] It is commonly forbidden to drive strips of sheet metal between plates to accomplish same result as split calking. The only proper way to remedy such trouble is to cut rivets out, lay the plates up well, and drive new rivets. In the trench each field joint may be tested by an apparatus which puts pressure on a single joint, but it has been found that rigid inspection is better than any test. Tap every rivet and try every part of both inside and outside calking with a *very* thin-bladed knife, or a machinist's "feeler."

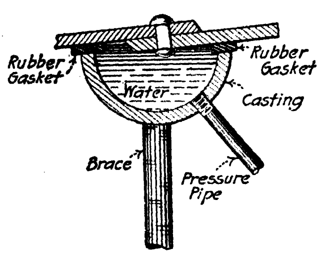


FIG. 142.—Apparatus for testing separate field joints (steel pipes).

Protection of Coating. Loading on, and unloading from, cars and transporting to the trench should be carefully specified and inspected. In one case unloading from cars was successfully accomplished by means of rope slings and a tall derrick. Two methods were used on one job in transporting from cars to the work; while snow lasted, one pipe at a time was put on a specially constructed low sled, and rolled off sideways at the proper place; when there was no snow, pipes were put on to a movable skid lying on rollers in a light truck; at the right location the skid and pipe were slid endwise from the wagon, and then the pipe rolled sideways from skid; both were successful. After

being strung along the trench,* two or three pipe sections are often riveted together; in case it becomes necessary to move them longitudinally, a successful method is to roll the pipe sideways onto skids and move the skids and pipe on rollers. Placing the pipe in the trench and connecting it is sure to mar the coating. In cold weather there is no trouble in making everybody who has to walk on or in the pipe wear rubber shoes or boots, but it is impossible to keep them from dropping tools. In warm weather it is impossible to make men wear rubbers or to keep men with hard boots off the coating, though, as the coating is more pliable at this time, not so much damage is done. Damage may also result from riveting gangs, dropping tools, sitting on boards whose edges cut into the coating, dragging kegs of rivets, etc.; also from stones in the backfilling. It is specified sometimes to use canvas for covering inside and outside of the pipe to protect the coating during the above operations, but this is impracticable; the pipe often has water in it, and the canvas would freeze stiff in winter; it cannot be laid across joints to be riveted, and often a large quantity would be required to protect all the pipe exposed, besides requiring much labor and inspection to see that the canvas is always spread.

Closure pieces are required where a valve must be at an exact station, or obstructions passed, or a curve have an exact location, since the "field" stationing does not agree with the plan, due to play at field joints amounting to several feet on long tangents. When an important station is approached, a gap is left for insertion of the closure, which is ordered 3 ft. longer than distance called for on plan. The closure has riveted longitudinal seams, shop rivets being omitted for 3 ft. on one end. This enables erectors to slip that end over pipe already placed, the other transverse joint being in proper position. Required position of transverse joint is then marked, extra length burned off in the field, holes for transverse joint drilled, closure put back in place, and riveting of transverse and longitudinal joints completed.

Twin Pipe Lines. Spacing center to center of a dual conduit is determined by the security required in case of a break in one line. The 11 ft. 3 in. steel-pipe siphons of the Catskill aqueduct are 45 ft. center to center. The 72-in. lines of the Jersey City conduit are 12 ft. center to center. The 60-in. raw-water lines at Cleveland are 8 ft. center to center, diverging to 10-ft. at cross-connections, which occur about every half mile.

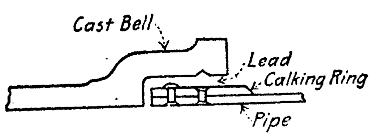
Earthwork. One foot each side of pipe affords space sufficient for riveting pipe, whether the trench is sheeted or not; many specifications stipulate payment over a width 3 ft. greater than internal diam. of pipe. For bell holes, an allowance of 1.5 ft. either side of pipe and 6 ft. long is sufficient. On recent work consisting of 4- and 6-ft. lock-bar pipe in 30-ft. lengths, where considerable time elapsed between laying and testing, bell holes were sheeted so that backfilling could be placed except at the bells; temperature effects were thus reduced. If the conduit is laid through swampy ground, carefully selected material should be borrowed for backfill, and the original excavated material disposed of. In borrowing material, allow liberally for wastage due to settlement in the swamp. One conduit which was laid on the swamp surface and surrounded by embanking to 3 ft. over the top, showed eventually

* In city streets, pipes stored along trenches must be guarded, to prevent children playing in them and damaging coating; on one job considerable repairing of coating was required due to miscreants cutting initials in the soft coating.

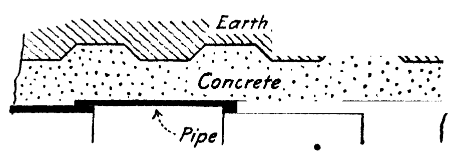
but 1 to 2.6 ft. over top. On the Jersey Meadows, it has been found that a more durable pipe results if excavation is restricted in depth so that the top surface of enmeshed roots and growths is not penetrated; below this layer lies deep muck with properties injurious to metals.

Earth-cover requirements differ in city and country; saving in excavation can be obtained in the city by locating on park lands wherever feasible. Provide adequate cover in country at railroad and highway crossings. Many railroads require pipe either to be placed in culvert, or surrounded with concrete, to assure that no settlement can occur to break the pipe, and interrupt traffic.

Catskill aqueduct had 3-ft. cover in the country (9- to 11-ft. diam.); and 4 ft. in city streets (66-in. conduit). Pipe lines in country for Ogden, Utah, and Brooklyn had 3-ft. cover; Jersey City conduit, 3-ft. cover; Weston aqueduct inverted siphon, cover of 2 ft., except one stretch, 2.5. Where embanked, top width was 10 ft. minimum; side slopes, $1\frac{3}{4}$ to 1.



Connection of steel pipe to valves or cast-iron pipe.



Anchorage of steel pipe in earth.

FIG. 143.

Erection in Place. Many penstocks and some water-supply pipes have been built up, plate by plate, in place. Plates are bent to shape and punched in the shop and shipped nested; this involves two longitudinal seams. Some makers prefer this procedure for diameters above 8 ft., and claim economy for it; 18-ft. penstocks for Ontario Power Co., Niagara Falls, were built in place from $\frac{1}{2}$-in. plates shipped flat, the average speed being 48 lin. ft. per day.[27] Freight is cheaper for plates than for pipes. At Trenton Falls, the erecting derrick traveled astride the 12-ft. pipe as it was laid, handling the next length into position by boom. The maximum progress per 9-hr. day was 22.5 ft.[28] In field erection, give each piece an erection mark.

Bends* are used to avoid obstructions and to change direction. On both riveted and lock-bar lines bends may be either riveted or cast-iron specials. On long lines, assume 5 per cent. of length as bends. If full-sized plates (7.5 and 15 ft.) are used, bends do not add greatly to the cost. On large conduits, special bends of short chords and short radius are generally sublet to pipe shops which specialize on difficult work. Estimate cost of bends and such special work generally at one-third more per ft. than straight pipe.

Bends fabricated of riveted steel are made similarly to old-fashioned stovepipe bends, *i.e.*, each chord of the bend is a cylinder truncated on one or both ends. The amount by which the shortest element in the cylinder is shorter than the longest element is known as the "cut." As a rule, for $\frac{3}{8}$-in. plate, a single cut should be no greater than 5 per cent. of the pipe diam.; this means a cut of 2.4 in. on a 48-in. pipe. Keep the cuts, or bevels, the same on each

* For anchorage, see p. 334.

bend, if feasible, as computations are simplified and errors in shop lessened. With $\frac{5}{16}$-in. plate, one chord of bend may be figured as subtending angle of 5°; $\frac{3}{8}$-in. plate, 4°; $\frac{7}{16}$-in. plate, 3°.

Radius of bend should be selected, if practicable, so that the longest element of each section measures a plate length (7.5 or 15 ft.). Usually conditions require a shorter radius; the minimum radius is fixed by requirement that minimum distance between gage lines of circular seams shall be 10 in. Short radii make work a little more difficult, and interfere with dipping. One bevel on every second sheet only results in a curve of 280-ft. radius, approximating a 20° railroad curve. No special advantage accrues from use of a standard radius. At one special location on the Bayonne 48-in. line, conditions necessitated a 6.5 ft. radius. On penstocks the Hydraulic Power Committee recommends the radius be no greater than 5 diam. of pipe, as this allows better provisions for anchoring; in waterworks practice there is not so great concern with anchorage problems, although attention should be paid them.

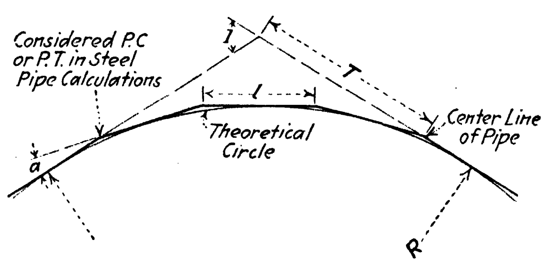


FIG. 144.—Alinement of steel pipes. Radius of curve.

In stationing short bends for a conduit, office and survey work are expedited if no correction is made for the curve length, *i.e.*, give stations for the P. C., P. I., and P. T. measured along the tangent.

To Find Radius of Curve (Bend) in One Plane. With equal angles at all angle points, center line of pipe lies entirely outside the theoretical curve, touching it only at the midpoints between angles; also the point of the first angle and the point of beginning of the theoretical curve are not coincident.* This must be taken into account in both designing and locating curves, ordinary railroad curve formulas not applying without modification. If l = distance in ft. between angle points; T = distance in ft. from intersection of tangents to first angle point; I = total deflection angle; a = deflection angle at each point; R = radius of theoretical curve; then, $R = \frac{l}{2}\cot\frac{1}{2}a$; $T = R\tan\frac{1}{2}I - \frac{l}{2} = \frac{l}{2}\cot\frac{1}{2}a\tan\frac{1}{2}I - \frac{l}{2}$. If field and office calculations are based on tangent measurements (see above), proceed as follows: Assume there is given only the deflection angle I of the tangents = 43°, pipe diam. 48 in., and thickness $\frac{5}{16}$ in. Inasmuch as no chord should subtend a greater angle than 5°, number of chords required = $\frac{43}{5}$ = 8+; use nine chords; $a = \frac{43}{9}$ = 4°46′. If plates are to be 7.5 ft. long, (R + 24 in. + $\frac{5}{16}$ in.) × tan. $\frac{4°46'}{2}$ =

* See Fig. 144.

3.75 ft. Therefore $R = 90.34$ ft. − 2.03 ft. = 88.31 ft. The bevel on each end of each pipe = 48 × tan. 2°23′ = 2.03. Call it 2 in. (Never give cuts closer than $\frac{1}{16}$ in.)

Bends in Two Planes. Pipe cuts may be so made as to result in the pipe being bent in two planes; wherever feasible, it simplifies matters if the bend in each reference plane is made separately. The following calculation is for first condition, using a combination of simple curves. Assume point S depressed 7 ft. (see Fig. 145). Lay curve 1 on the regular grade; rotate curve 2 on the tangent of its axis, at A, the point of beginning of curve, so that its end B drops half the vertical distance required; rotate curve 3 an equal amount in the opposite direction so as to be tangent to curve 2 at B and have the tangent at S horizontal. Lay 4 like 3, 5 like 2, and 6 like 1. Computation of curves 1 and 6: Total given or assumed angle = 44°; make a = 4° (a is angle between adjoining beveled sheets); first and last angles = $\frac{1}{2}a$. Assume 7-ft. sheets, 48-in. pipe, radius R = 94.59 ft., using full sheets and no unbeveled straight sheets between angle points. Make curves 2, 3, 4, 5 of same radius and in same manner, *i.e.*, as chords to the theoretical

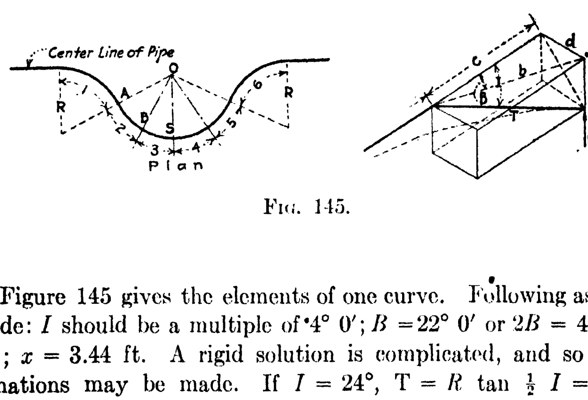


FIG. 145.

curve. Figure 145 gives the elements of one curve. Following assumptions were made: I should be a multiple of 4° 0′; B = 22° 0′ or $2B$ = 44° 0′; R = 94.59 ft.; x = 3.44 ft. A rigid solution is complicated, and so successive approximations may be made. If I = 24°, T = R tan $\frac{1}{2}$ I = 20.11 ft.; $b = \sqrt{T^2 - x^2}$ = 19.81 ft.; $c = T \cos I$ = 18.37 ft., and $\cos B = \frac{c}{d}$, B = 21° 59′ 41″; $2B$ = 43° 59′ 22″. See also p. 408. Some deflection can be gained in the field due to play in the rivet holes; on a 48-in. pipe, $\frac{3}{8}$ in. thick, 3 in. have been made in a 30-ft. length.

Pipe-line Testing. Special precautions should be taken for hydrostatic pressure tests on a steel pipe line, prior to placing it in service. (1) See that trench has been refilled between "bell" holes to prevent floating of pipe. (2) All test heads or gate valves should be securely braced. (3) All blow-off valves should be closed. (4) No section of pipe should be tested without having at least one automatic air valve installed and in proper working order, even though it may not be required for permanent operation. Open wide the gate valve, between the steel pipe line and the air value.[29] Manufacturers furnish or rent movable bulkheads, one type of which is shown in *E. C.*, Oct. 20, 1915, p. 317. Where sectionalizing valves are installed at mile intervals, they are a convenience for testing.

STEEL-PIPE DETAILS

Plugs. For passing hot rivets into pipe, holes are made in plates next to field joints, $1\frac{1}{4}$-in. diam. for $\frac{3}{4}$-in. rivets, to $2\frac{1}{2}$-in. diam. for $1\frac{1}{4}$-in. rivets; these holes are ultimately tapped and closed with steam-fitters' screw plugs with tight-fitting threads; the plugs should not project far into the pipe. Wrought-iron screw plugs used in rivet-passing holes have given no trouble, beyond a little leakage easily repaired by calking; cast-iron plugs are poor practice. Plugs of form shown in Fig. 146 were used on Catskill aqueduct pipes. Because of slight yield in compression when screwed in and action of water pressure when in service, they are tighter. Threads must fit tightly. Soft lead grommets or washers are sometimes used under shoulders of plugs and squeezed tightly against pipe plate. Openings in $\frac{7}{16}$- and $\frac{1}{2}$-in. plates should be reinforced by plates.

Air Valves. See p. 446.

Manholes on Catskill aqueduct siphons are cast steel. For common sizes of pipe access manholes of boiler type (Fig. 147) are used; they are usually riveted on in the shop. A common spacing is every 1000 ft., and at the foot of a steep slope; 600 to 800 ft. was used on raw-

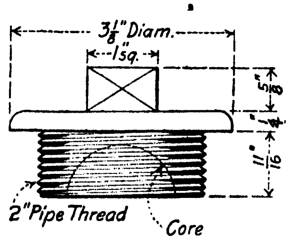


FIG. 146.

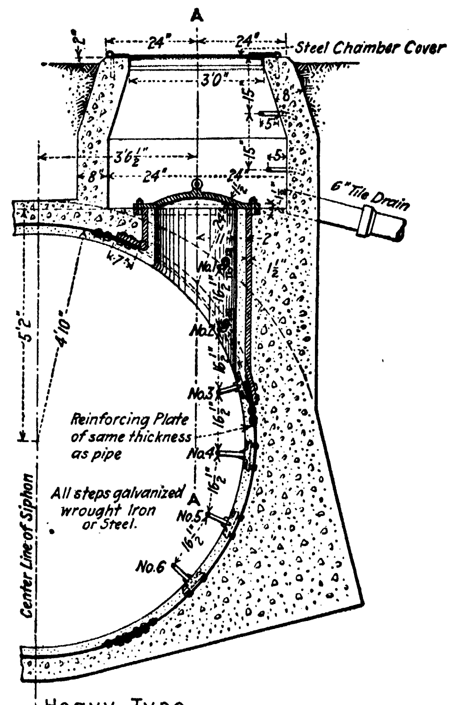


FIG. 147.—Manhole, teel pipe siphons, Catskill aqueduct.

water conduit, Cleveland. Insert a thin plate for calking between a heavy manhole casting and the pipe. On welded pipe, manhole and other saddles are welded on.

Connections. If a connection for a small blow-off, air valve,* etc. must have an exact stationing, saddles (similar to Fig. 155) are often fitted in the field, openings being made by an oxyacetylene torch. On Los Angeles aqueduct, 4-in. couplings[30] for irrigation connections are welded to the steel pipe. Large connections require use of closure pieces to assure correct stationing. Blow-offs, if not free draining, require a well from which water

* For design, see p. 446.

can be pumped. At chambers or connections with cast-iron pipes, steel pipes should be reinforced with a band against which to calk the lead joint; this band should be riveted to the pipe with the heads in the row farther from the end of the pipe countersunk on both sides and the end row countersunk on the inside only, the button heads furnishing a grip for the lead. Depth of bells for such connections should be 6 or 7 in., but 5½ in. may suffice for small pipes. Lead joints of bell-and-spigot type should be used for connecting steel pipes to valves, Venturi meters, and similar castings wherever contraction of the pipes would otherwise break the castings. For flange connections to specials and valves in a steel pipe line use rolled, cast, or forged steel flanges—not cast iron.

Gate valves in line are generally of the two-bell type, a reinforcing ring being riveted outside the steel pipe to strengthen it against calking and to retain the lead. Reducers are often used either side of the valve (see also p. 437).

Anchorages are provided in penstocks to take the unbalanced thrust of water flowing around bends. Analysis for design is given in 1923 report of Hydraulic Power Committee.* If pipe is buried, the question of anchorage is not so important, although some provision should be made so that pipe will not pull out at valves, or other calked joints. Steel pipes should be reinforced and anchored with concrete at gate chambers, for 20 to 30 ft. Curves, if of small deflection, need not be anchored. Sides and bottoms of trenches in firm earth may be notched, to save concrete and increase the strength of the anchorage.

Steel Pipe as Bridge. A pipe subject to bursting pressure and used at the same time as a beam or bridge should be figured so that the maximum stress resulting from the two conditions will not exceed the allowable stress. If, comparing the total weight of the beam (pipe and contained water) with the safe strength, the thickness is not sufficient for beam strength, rather than increase thickness, calculate body of pipe for bursting pressure only, and rely on longitudinal members (angles, tees, etc.) riveted to top and bottom for beam action. In determining thickness, allow liberally for rusting and deterioration in such exposed places as pipe bridges are usually built in. Two 8-ft. steel pipes, supported on masonry piers, were used for stream control at Olive Bridge dam for 2 years, without cover. There was one 42-ft. span with full pipe, one 47-ft. span half full; when removing the pipes, one span of 75 ft. occurred for 1 day with pipe one-quarter full; no deflections or deformations in the pipes were observed. These pipes had two double-riveted longitudinal seams, single-riveted ring seams 7.5 ft. apart, and plates $\frac{7}{16}$ in. thick. Welded pipe manufacturers claim that 40-ft. span is possible.

WROUGHT-IRON RIVETED PIPE

Durability of Wrought-iron Plates. Wrought-iron pipes in California 50 years or more old are in good condition. Tests of wrought-iron and steel cups filled with water and cinders showed loss by corrosion, in the former, of 40 per cent. and in the latter of 89 per cent.† Sheet-metal mills call for wrought

* N. E. L. A., O. G. Thurlow, Chairman.
† See also p. 812.

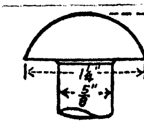
iron plates, rather than their own steel plates, for annealing boxes. Wrought-iron roofing has lasted 13 years alongside steel, which lasted only 3 years. Iron smokestacks last 15 years, steel 3. Cost of riveted wrought-iron pipe is about 1.33 times price of steel. Sheet-iron pipes are used by East Bay Water Co. in sizes 12 to 20 in.[32]

Specifications.* These specifications are given at length because of the care with which they were prepared by the company's former chief engineer, Herrmann Schussler, and the successful results achieved. First used in 1903 by Spring Valley Water Co., San Francisco, for 54-in. Alameda pipe, and still used in 1925.

Manufacture of Flanged Boiler Iron. Take charcoal iron blooms rolled into bars about 14 in. wide and 3 to 4 ft. long, about 1 in. thick, for bottom of pile. This bloom to be made of iron only, melted in charcoal fire and hammered into a bloom, then rolled into above wide flat bar. On this slab, bars and best scrap, both of charcoal iron, are closely and evenly packed crosswise and with flush or even top. On top of this pile, another 3- to 4-ft. by 14-in. by 1-in. slab, as above, is placed, making a pile 8 to 10 in. thick, or high. Thereupon this pile is heated to welding heat, put through rolls, and welded down to 1½-in. thickness, more or less, *across* grain of cover. Then pile is reheated and rolled out to width of finished plate *with* grain of cover. Then slab is quickly turned at right angles and rolled against grain of cover to full length of finished plate. Scrap from shearing plates goes to make up next piles. Tensile strength shall be between 48,000 and 50,000 lb. per sq. in lengthwise with plate, 2 per cent. variation allowed below lower and above upper limit. Tensile strength crosswise with plate shall not be less than 40,000 lb. Elastic limit about 32,000 and not less than 30,000 lb. per sq in. Elongation 15 to 20 per cent. in 8-in. section. Reduction of area not less than 25 per cent. Thickness shall not vary more than 3 per cent. above or below given standard. Each plate shall be sheared true to dimensions, with sides and ends straight and rectangular to each other. Shipping weight of plates shall not be more than 8 per cent. above nor 3 per cent below required weight of plate computed at 40 lb. per sq. ft. 1 in. thick, and thickness of plate at sheared edges shall not be less than 97 per cent. of thickness specified. Each plate shall be flat, smooth, and even, of even thickness throughout, and free from warping, buckles, cracks, splits, flaws, rust, and all other defects. The iron shall be of American manufacture, close-grained, tough, and thoroughly pliable while cold, also allowing cold scarfing to a fine edge at laps, without splitting or cracking, and shall not crack between rivet holes, between holes and edge of plate while being rolled, or while rivets are being driven. All plates showing flaws of any kind or exhibiting a hard and brittle character, and that do not in every way meet above requirements, will be rejected. Plates may vary *in width* $\frac{5}{16}$ in., but shall be not less than $\frac{1}{16}$ in. less than 60 in., nor more than 60¼ in. *In length*, they shall be not less than ⅛ in. below nor more than ¼ in. above lengths specified. Contractor shall be responsible for storing plates under a tight roof in mill premises and in tight box cars, without exposure to moisture of any kind.

Making 54-in. Pipe. Plates and rivets of which pipe is to be made will be delivered at place of manufacture in San Francisco. The following table shows dimensions of plates and rivets.

* Comm. 12, *A W W A*, is preparing specifications

Plates	Thickness, in.	Weights, lbs. per sq. ft.		Size of plate, in.
	$\frac{9}{32}$ $\frac{9}{32}$	11 11		58×175 58×176½
Rivets		Size of body, in.		Rivet head
		Diam.	Length	
	Seam Lap	$\frac{5}{8}$ $\frac{5}{8}$	$1\frac{5}{8}$ $1\frac{1}{4}$	

During manufacture of pipes, all plates and rivets shall be kept under roof and cover, and in no way exposed to rain or fogs. Pipe is to consist of large and small straight courses. Plates for large courses, 58 in. wide by 176½ in., are to be trimmed to exact size, with opposite sides parallel to each other and corners rectangular. Plates for small courses, 58 in. wide by 175 in., are to be trimmed so that when punched and riveted into courses they fit tightly in large courses. Rivet holes are to be punched as follows: Center to center in each row of straight seams, 2¼ in.; center to center between two rows of straight seams, 1¼ in.; center to center, round seam, 1⅞ in.; center of seam to edge of sheet, 1¼ in.; diam. of rivet holes, $\frac{11}{16}$ in. Where, at end of each course, lap falls between two thicknesses of iron, plates shall be drawn to a fine edge, through which edge, upon riveting courses together, one rivet of round seam shall be driven to insure absolute tightness. Each plate shall be rolled to a perfect cylinder of required diam. All punching of plates shall be done by automatic and accurate multiple punching machine and not through a frame or templet—opposite lines of rivet holes to be absolutely straight and parallel to each other and spaces between respective holes evenly divided.

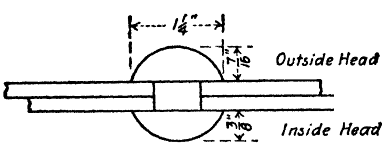


FIG. 148.

All riveting in shop shall be done with hot rivets and by hydraulic machinery, exerting a slow pressure of not less than 20,000 lb. on each rivet head. Prior to driving rivet, plates shall be pressed together by a slow squeeze from same machine. Head of rivet on inside and outside of pipe is to be formed by a cup in the die, and inside rivet head not to be higher than ⅜ in. and to receive, as nearly as possible, shape shown in Fig. 148, while outside head may be $\frac{7}{16}$ in. high in center.

At each junction of straight and round seam where three thicknesses of iron come together, lap rivets are to be used. Straight seams are to be all on one side of each length of pipe, alternating to right and left not more than 1 ft. Each pipe length shall consist of six large and small straight courses, with a large course at one end and a small course at other. All round and straight seams and laps are to be thoroughly split and calked in first-class boiler-works fashion, while for 4 in. from all laps all seams are to be chipped and calked. S. V. W. W. may test any length with water pressure up to 200 lb. at expense of contractor. Pipe must be absolutely tight under pressure of 200 lb. per sq. in. Small course shall be either chipped or ground with emery wheel to bevel of 45°, so as to decrease friction of water in pipes. All punching, riveting, calking, etc. shall be done with best workmanship, so as to insure strength and absolute tightness. S. V. W. W. is to dip the pipe in asphaltum coating and to transport to and distribute along ditch. Space for asphaltum kettles and tanks, and rig and power for handling pipe while

being dipped, and ample room for dry storage of pipe before dipping and for open air storage after dipping, shall be furnished by contractor.

Laying Pipe. Pipe ditch and necessary joint holes will be dug by S. V. W. W. Pipes are to be connected in ditch by inserting small course of one pipe into large course of other. Where pipe curves, strap joints are to be used, iron and rivets necessary for such joints to be furnished by S. V. W. W., strap iron being 8 in. in width and to be put on outside of pipe in two sections, with straight seams on sides, so as to form joints as perfectly as those required to be made in shop. Contractor is to roll, shear, punch, and fit bands, and employ best of workmanship in this work and in scarfing, riveting, splitting, chipping, and calking and other necessary work, as is required for pipe. The two straight seams of strap shall be chipped and calked, as well as round seam, for 4 in. on each side of laps. Where straps are used, lengths of pipe are to be so placed in ditch that ends of pipe butt together, or nearly so. Distance between ends of pipes at such bends, in widest part, is not to be more than 3 in. Before straps are fitted and riveted to ends of pipe, these ends are to be carefully scraped and entirely cleaned of coating for 3 in. from each end, both

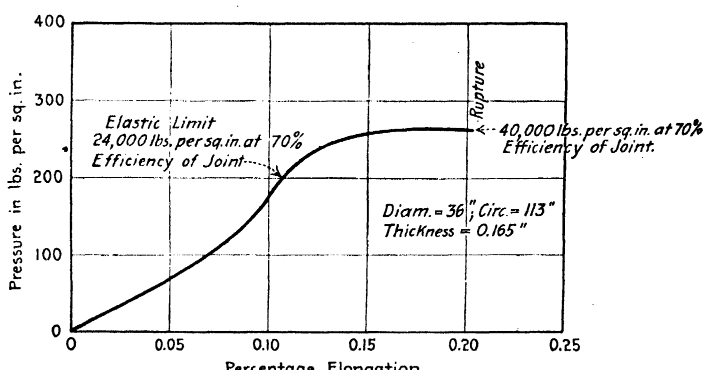


FIG. 149.

in and outside of pipe, same as all other joints made in ditch, so that strap joints as well as other joints make a perfect union of iron to iron. Round seams of pipe, as well as seams of strap joints, shall be riveted with hot rivets, forming good, substantial heads, both in and outside of pipe, of shape and proportions shown on p. 336, care being taken that a rivet is placed through scarfed edges of plate at all laps, for which laps lap rivets are to be used as specified for pipe. Where curvature of pipes is so great that above strap joints are insufficient to make pipe follow such curves, same is to be accomplished by inserting one or more single courses, or such single courses intermingled with lengths of pipe. Contractor is to furnish manholes, blow-outs, and air valves, and connect them with pipe in such places as chief engineer directs, each fitting being provided with a wrought-iron ring $\frac{5}{16} \times 3$ in. on inside, hot rivets passing through it and iron of pipe and flanges of fitting—only hot rivets being used; edge of plate shall be chipped and calked against inside face of fitting; rivet joints, as well as apparatus so attached, shall be perfectly watertight. Any plates or rivets injured or allowed to rust, or otherwise damaged by contractor while in his keeping, and before acceptance and dipping by S. V. W. W. shall be replaced at his expense.

Contractor shall provide dry storage for iron and rivets, so as to keep moisture, rain, fog, etc., entirely away from them. Contractor is required to manufacture, ready for dipping and transportation, for every working day after arrival of

iron, not less than ft. of pipe, and is also required to lay and connect the same, after it has been dipped and transported by the S. V. W. W., in manner described and specified, at same rate per day as it was manufactured.

Tests of Strength and Stretch. Engineers of Spring Valley W. W., San Francisco, tested (June, 1911) 36-in. riveted wrought-iron pipe made under specifications like above, with results shown in Fig. 149. Elongation of the iron or circumferential stretch of the pipe was measured by a tape wrapped around the pipe and held by special apparatus.

INGOT IRON PIPES

"Armco" Ingot iron resists corrosion through the elimination of impurities in its manufacture; the total of impurities is claimed as less than 0.16 per cent. Gaseous inclusions are reduced to the minimum. Since manganese is known to accelerate corrosion, aluminum is substituted for eliminating the iron oxide in the metallurgy; it serves as a powerful deoxidizing and degassifying agent. This degassification produces a dense and nearly solid structure. "Armco" Ingot iron has free welding powers.

In Uncompahgre Valley, Colorado, U. S. Reclamation Service, in 1911, built a 26-in. pipe of "Ingot" iron sheets, since the alkali soils along the line have a strong corrosive action on ordinary iron and steel. This is believed to be the first extensive use for pipe lines. Specifications called for following properties: ultimate tensile strength, not less than 48,000 nor more than 52,000 lb. per sq. in.; elastic limit, not less than 35,000; carbon, not more than 0.01 per cent.; manganese, 0.02; phosphorus, 0.005; sulphur, 0.02; oxygen, 0.03 per cent.; and trace of silicon.[33] Correspondence shows that there has been no detailed inspection since putting into service, and that only repair up to 1921 has been minor repainting. "Has stood up in excellent manner."

Table 81. Results of 28 Tests of "Ingot" Iron Plates

	Max.	Min.	Ave.
Elastic limit, lbs. per sq. in.	46,750	22,800	34,840
Ultimate strength, lbs. per sq. in.	59,000	40,580	49,230
Elongation in 8 in., per cent	37	10	26
Reduction of area, per cent	88	40	68
Carbon, per cent	0.02	0.01	0.012
Manganese, per cent	0.02	0.01	0.013
Phosphorus, per cent	0.011	0.001	0.0053
Sulphur, per cent	0.017	0.022	0.02
Oxygen, per cent	0.037	0.003	0.022

SPIRAL-RIVETED PIPE

Stresses in spiral-riveted steel pipe may be derived by the following formulas:[34]

(*A*) To find unit stress, S, lb. per sq. in. in the plate, $S = \frac{DP}{4t}(2 - \sin^2 \theta)$.

D, pipe diam., in in.; P, internal pressure, lb. per sq. in.; t, thickness of plate, in in.; θ, the angle made by a tangent to the joint at the plane of horizontal axis, with this axis. When $\theta = 0$, the spiral joint becomes the ordinary

longitudinal joint, and formula reduces to the usual pipe formula: $S = \frac{DP}{2t}$.

(*B*) To find θ for a given riveted joint efficiency of such magnitude that the strength of the joint will be equal to that of the plate for the longitudinal section:

$$\sin \theta = \sqrt{2 - \left(\frac{e}{50}\right)},$$

e is the efficiency of the joint, in per cent. θ may be found by measuring the pitch p, *i.e.*, the distance between spiral joints. Then $\tan \theta = \pi \frac{D}{p}$.

In manufacturing* spiral-riveted pipe, a strip of sheet steel is wound into helical shape with one edge overlapping the other for riveting; the sheet is drawn and formed in such manner that metal-to-metal contact in spiral seam is obtained, stretching steel on outer lap, slightly offset, in order that pipe may be more nearly smooth on inside. Riveting is done cold by compression, not by hammering, thus insuring complete filling of holes with slight countersink. Pipe comes from machines in a continuous piece, and is cut to any desired length. It is manufactured in sizes from 3 to 40 in., of various thicknesses (see Table 82). The patents of Abendroth and Stein of New York expired several years ago. Each length of pipe, when pressure is specified, is tested to 50 per cent. more than the working pressure before shipping.

Advantages. Spiral-riveted pipe will stand a greater collapsing pressure than other steel pipe of same thickness. The wide lap of the helical seams gives great strength for spanning ravines or withstanding heavy earth fills. Spiral riveted pipe, 6 in. diam., 24 ft. long (No. 12 gage, double-extra heavy), has sustained, as beam supported at ends, a toal distributed weight of about 2900 lb.[35] The pipe is easily erected. Spiral-riveted pipe is used extensively in waterworks and hydro-electric plants for pressure up to 400 lb., for hydraulic mining under heaviest pressure, in paper and pulp mills, for suction and discharge on centrifugal pumps, in feed-water heaters, and for exhaust-steam purposes; especially adapted for long lines, where strength, durability and price enter into consideration. Said to be 25 per cent. cheaper than cast iron for sizes under 10 in., and 35 per cent. for sizes 12 to 20 in. This depends upon local conditions.

Experience. Many pipes, galvanized or protected by mineral-rubber composition, have been in use over 20 years, and are still in good condition. Pipes of No. 16 galvanized steel have been in use in small water systems since 1904. In San Gabriel, Cal., pipes under 35-lb. pressure have given no trouble. Lexington, Mo., pipe was laid in 1887. Spiral pipes at Portsmouth and Gorham, N. H., Brattleboro, Vt., Langerville, Dixfield, Millinocket, and Lewisport, Maine, were laid about 1902. A 24-in. spiral pipe, No. 12 gage, laid on Long Island in 1887, was uncovered in 1907, apparently in good condition; in addition to tar dip, it was wrapped spirally with tar paper while coating was hot. Pressures in this pipe are light.

Field Joints† and Specials. Pipe should be laid with the laps in direction of flow. Shop lengths can be of any length up to 30 ft.; they are equipped

* Makers include Abendroth and Root, Newburgh, N. Y.; American Spiral Pipe Works, Chicago; Robertson Bros. Mfg. Co.; and Crane Co., Chicago, Ill.

† See also "Joints for Wrought-iron and Steel Pipe," by R. S. Lord, *J. Eng. Soc. Western Pa.* 1915.

with some form of flange to facilitate erection (see Tables 83 to 88), or a *sleeve joint* of the Dresser type, shown in Fig. 150, is used. The Dresser joint allows slight deflections, and is claimed to take care of all expansion and contraction. It can be connected rapidly in the field; it is well adapted to pipe laid above ground. The slip joint (Fig. 150) is used largely for medium and low-pressure work; the sleeve, which is attached to one end of pipe, is wrapped with burlap or canvas soaked in red lead or liquid asphaltum, then driven into the

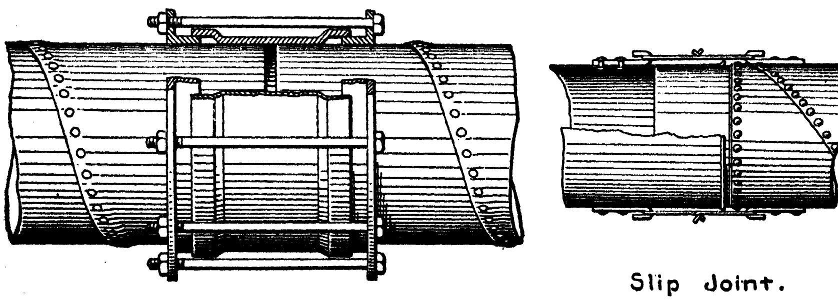


FIG. 150.—Joints for spiral-riveted pipes.*

adjoining pipe; the lugs are then connected by wire in order to hold the pipe securely.

Flanges.—On spiral-riveted and welded pipe forged steel flanges may be used. They are attached in various ways, dependent on the pressures to be resisted, and the degree of watertightness required. Tables 83 to 88 give the important data for several types. Table 100, page 390 gives corresponding data for cast-iron flanges cast on cast-iron pipe. In measuring for flanged pipes, allow $\frac{1}{16}''$ at every joint for the space occupied by the gasket. Always specify the type of drilling required: *i.e.*, straddle center or on center. In estimating weights, allow for bolts and nuts.

Lap joints using extra-heavy high-hub flanges have met with favor for high-pressure work; extensively used in large power houses; made by heating end of pipe and turning it over the flange, then facing end of pipe; no possible place for leakage, except through gasket; flange is loose and can be turned to any position for alinement of bolts. This joint, with slight modifications, is known under various trade names: Van Stone, Kellog Improved, Atwood, Cranelap, Walmanco, Pittsburgh, Mitchel Recessed Lap, Climax Rolled, and Whitlock Patented Joint.

Shrunk and peened joint (Fig. 151), also satisfactory, is made by boring the flange a little smaller than outside of pipe and then shrinking the flange on while hot; end of pipe is usually peened into flange by hand hammer or expanding machine. Some engineers require large sizes riveted, in addition to peening. Considerable strain is put upon the flange by the shrinking and any other than forged steel flanges are unsafe. This joint is also standard for U. S. Navy.

Table 82. Spiral Riveted Pressure Pipe[35]

Inside diam., inches	Thickness, U. S. Standard gage	Approx. weight, pounds per foot, asphalted	Approx. bursting strength, lb. per sq. in.
①	②	③	④
3	20a	1.9	1500
	18b	2.3	2000
4	20	2.4	1125
	18a	3.0	1500
	16b	3.7	1875
5	20	2.9	900
	18a	3.7	1200
	16b	4.5	1500
6	18	4.3	1000
	16a	5.3	1250
	14b	6.6	1560
	12c	9.2	2170
7	18	5.1	860
	16a	6.2	1070
	14b	7.7	1340
	12c	10.7	1860
8	18	5.8	750
	16a	7.1	935
	14b	8.8	1170
	12c	12.3	1640
9	16a	8.0	835
	14b	9.9	1045
	12c	13.9	1460
10	16a	8.8	750
	14b	11.0	935
	12c	15.3	1310
11	16a	9.7	680
	14b	12.0	850
	12c	16.6	1200
12	16a	10.6	625
	14b	13.0	780
	12c	18.2	1080
	10	22.5	1410

Inside diam., inches	Thickness, U. S. Standard gage	Approx. weight, pounds per foot, asphalted	Approx. bursting strength, lb. per sq. in.
①	②	③	④
13	16a	11.4	575
	14b	14.1	720
	12c	19.7	1010
	10	24.5	1295
14	16	12.9	535
	14a	15.9	670
	12b	22.2	940
	10c	27.6	1210
15	14a	17.0	625
	12b	23.7	875
	10c	29.6	1125
16	14a	18.1	585
	12b	25.2	820
	10c	31.5	1050
	8	38.1	1290
	6	44.7	1520
	3	51.6	1880
18	14a	19.9	520
	12b	27.6	730
	10c	34.5	940
	8	41.6	1140
	6	49.0	1360
	3	59.2	1660
20	14a	22.1	470
	12b	30.6	660
	10c	38.3	840
	8	46.2	1030
	6	54.1	1220
	3	65.6	1500
22	14	24.4	425
	12a	33.7	595
	10b	42.2	765
	8c	50.8	940
	6	59.5	1108
	3	72.2	1364
24	14	26.4	390
	12a	36.5	540
	10b	45.7	705
	8c	55.2	820
	6	64.6	1015
	3	78.4	1250

Inside diam., inches	Thickness, U. S. Standard gage	Approx. weight, pounds per foot, asphalted	Approx. bursting strength, lb. per sq. in.
①	②	③	④
26	12a	39.5	505
	10b	49.5	650
	8c	59.8	795
	6	70.0	935
	3	84.9	1154
28	12	42.1	470
	10a	51.7	605
	8b	63.6	735
	6c	76.6	870
	3	90.4	1071
30	12	45.3	435
	10a	56.8	560
	8b	68.7	685
	6c	80.5	810
	3	97.7	1000
32	12	49.1	410
	10a	61.6	525
	8b	74.3	645
	6c	87.1	760
	3	105.8	940
34	12	52.1	380
	10a	65.4	490
	8b	78.8	600
	6c	93.6	715
	3	112.3	880
36	12	55.1	365
	10a	69.1	470
	8b	83.4	570
	6c	97.8	680
	3	118.8	830
40	12	61.1	330
	10a	76.7	420
	8b	92.4	515
	6c	108.5	610
	3	131.8	750

a Standard thickness for spiral riveted pipe. *b* Extra heavy. *c* Double extra heavy.

Working pressure should not be more than 25 per cent. of ultimate strength or bursting pressure: *i.e.*, factor of safety should be 4 when working pressure is practically uniform; for pumping pressures, or where there is probability of water-hammer, larger safety factor is advisable. Galvanized pipe is furnished in any lengths up to 20 ft., and asphalted pipe up to 30 ft.

Nuts and bolt heads, by U. S. standard, should have the following dimensions, for both hexagon and square nuts:

Short diameter of rough nut = $1\frac{1}{2} \times$ diameter of bolt + $\frac{1}{8}$ in.

Short diameter of finished nut = $1\frac{1}{2} \times$ diameter of bolt + $\frac{1}{16}$ in.

Thickness of rough nut = diameter of bolt.

Thickness of finished nut = diameter of bolt − $\frac{1}{16}$ in.

Short diameter of rough head = $1\frac{1}{2} \times$ diameter of bolt + $\frac{1}{8}$ in.

Short diameter of finished head = $1\frac{1}{2} \times$ diameter of bolt + $\frac{1}{16}$ in.

Thickness of rough head = $\frac{1}{2}$ short diameter of head.

Thickness of finished head = diameter of bolt − $\frac{1}{16}$ in.

The long diameter of a hexagon nut may be obtained by multiplying the short diameter by 1.155 and the long diameter of a square nut by multiplying the short diameter by 1.414.

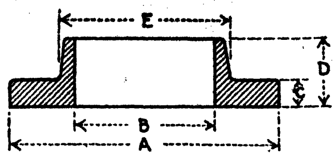


FIG. 151.—Shrink flange (Table 83).

Table 83. Standard (A. S. M. E.) Shrink Flanges, Forged and Rolled Steel[35]
(Dimensions in Inches)

Nominal diam.	Outside diam. A	Bore diam. B	Thickness C	Depth of hub D	Diam. of hub E	No. of bolts	Size of bolts	Diam. bolt circle
①	②	③	④	⑤	⑥	⑦	⑧	⑨
4	9	4 3/8	15/16	2 3/16	5 3/4	4	3/4	7 1/2
4 1/2	9 1/4	4 7/8	15/16	2 1/4	6 1/8	8	3/4	7 3/4
5	10	5 7/16	15/16	2 5/16	6 7/8	8	3/4	8 1/2
6	11	6 1/2	1	2 7/16	7 7/8	8	3/4	9 1/2
7	12 1/2	7 1/2	1 1/16	2 1/2	9	8	3/4	10 3/4
8	13 1/2	8 1/2	1 1/8	2 5/8	10	8	3/4	11 3/4
9	15	9 1/2	1 1/8	2 3/4	11 1/8	12	3/4	13 1/4
10	16	10 5/8	1 3/16	3	12 1/4	12	7/8	14 1/4
12	19	12 5/8	1 1/4	3 3/8	14 1/2	12	7/8	17
14	21	13 7/8	1 3/8	3 3/8	15 7/8	12	1	18 3/4
15	22 1/4	14 7/8	1 3/8	3 1/2	16 7/8	16	1	20
16	23 1/2	15 7/8	1 7/16	3 5/8	18	16	1	21 1/4
18	25	17 7/8	1 9/16	3 7/8	20 1/4	16	1 1/8	22 3/4
20	27 1/2	19 7/8	1 11/16	4 1/8	22 1/4	20	1 1/8	25

Shrink flange with deep hub is made to comply with A. S. M. E. standard for cast-iron flanges. They are made from best-quality open-hearth steel, and may be shrunk and peened on heavy pipe without danger of cracking. These flanges make reliable joints for medium- and high-pressure work. Flanges are furnished smooth forged to above dimensions except allowance on face for finishing. See more extensive table, p. 391 and 392.

Table 84. Extra Heavy High Hub Flanges, Forged and Rolled Steel
(Dimensions in Inches)

Nominal size	Outside diam. A	Bore, diam. B	Thickness C	Depth of hub D	Diam. of hub E	No. of bolts	Size of bolts	Diam. bolt circle
4	10	4 3/8	1 1/8	3 1/8	5 3/4	8	3/4	7 7/8
4 1/2	10 1/2	4 7/8	1 1/4	3 1/4	6 1/4	8	3/4	8 1/2
5	11	5 7/16	1 1/4	3 1/4	7	8	3/4	9 1/4
6	12 1/2	6 1/2	1 1/4	3 1/4	7 15/16	12	3/4	10 5/8
7	14	7 1/2	1 5/16	3 3/8	9 1/4	12	3/4	11 7/8
8	15	8 1/2	1 3/8	3 1/2	10 1/16	12	7/8	13
9	16	9 1/2	1 7/16	3 5/8	11 1/8	12	7/8	14
10	17 1/2	10 5/8	1 1/2	3 3/4	12 1/4	16	7/8	15 1/4
11	18 3/4	11 5/8	1 9/16	3 7/8	13 3/8			
12	20	12 5/8	1 5/8	4	14 1/4	16	7/8	17 3/4
14	22 1/2	13 7/8	1 3/4	4 1/8	16 1/16	20	7/8	20
15	23 1/2	14 7/8	1 13/16	4 1/4	17 1/2	20	1	21
16	25	15 7/8	1 7/8	4 1/2	18 1/2	20	1	22 1/2
18	27	17 7/8	2	5	20 3/4	24	1	24 3/4
20	29 1/2	19 7/8	2 1/4	5 1/2	22 1/2	24	1 1/8	26 3/4

Flanges are furnished smooth forged to above dimensions, except allowance on face for finishing. Thickness may be increased as desired. High-hub flanges are not suited for threading, as bore is too large.

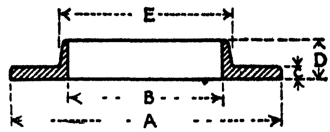


FIG. 152.—Flange for riveted steel pipe (Table 85).

Table 85. Forged Steel Flanges for Riveted Pipe
(Dimensions in Inches)
A S M E Standard

Nominal size, in	Outside diam A	Actual bore B	Thickness C	Depth of hub D	Diam of hub E	No of bolts	Size of bolts	Diam bolt circle
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
4	9	4 3/16	9/16	1 9/16	4 13/16	8	5/8	7 1/2
5	10	5 3/16	9/16	1 11/16	5 7/8	8	3/4	8 1/2
6	11	6 3/16	9/16	1 11/16	6 7/8	8	3/4	9 1/2
7	12 1/2	7 3/16	9/16	1 13/16	7 7/8	8	3/4	10 3/4
8	13 1/2	8 3/16	5/8	1 7/8	8 7/8	8	3/4	11 3/4
9	15	9 1/4	5/8	1 7/8	9 15/16	12	3/4	13 1/4
10	16	10 1/4	11/16	1 15/16	10 15/16	12	7/8	14 1/4
12	19	12 1/4	3/4	2	12 15/16	12	7/8	17
14	21	14 1/4	3/4	2 1/8	15 1/16	12	1	18 3/4
16	23 1/2	16 1/4	3/4	2 1/4	17	16	1	21 1/4
18	25	18 5/16	3/4	2 1/4	19 1/16	16	1 1/8	22 3/4
20	27 1/2	20 5/16	7/8	3 1/8	21 1/16	20	1 1/8	25
22	29 1/2	22 3/8	7/8	3 1/8	23 3/16	20	1 1/4	27 1/4
24	32	24 3/8	7/8	3 1/8	25 3/16	20	1 1/4	29 1/2

Larger sizes can be furnished on special order When quotations are wanted on flanges with holes in hub for rivets state size and spacing Flanges are smooth forged to above dimensions, shipped plain without holes unless ordered otherwise

Riveted Pipe Manufacturers' Standard

Nominal size, in	Outside diam A	Actual bore B	Thickness C	Depth of hub D	Diam of hub E	No of bolts	Size of bolts	Diam bolt circle
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
3	6	3 3/16	5/16	1 1/2	3 9/16	4	7/16	4 3/4
4	7	4 3/16	5/16	1 1/2	4 9/16	8	7/16	5 15/16
5	8	5 3/16	5/16	1 1/2	5 9/16	8	7/16	6 15/16
6	9	6 3/16	3/8	1 1/2	6 9/16	8	1/2	7 7/8
7	10	7 3/16	3/8	1 5/8	7 9/16	8	1/2	9
8	11	8 3/16	3/8	1 3/4	8 5/8	8	1/2	10
9	13	9 1/4	3/8	1 3/4	9 11/16	8	1/2	11 1/4
10	14	10 1/4	3/8	1 7/8	10 3/4	8	1/2	12 1/4
11	15	11 1/4	7/16	1 7/8	11 3/4	12	1/2	13 3/4
12	16	12 1/4	7/16	1 7/8	12 3/4	12	1/2	14 1/4
13	17	13 1/4	7/16	2	13 3/4	12	1/2	15 1/4
14	18	14 1/4	7/16	2	14 3/4	12	1/2	16 1/4
15	19	15 1/4	9/16	2	15 3/4	12	1/2	17 7/16
16	21 1/2	16 1/4	5/8	2 1/4	16 3/4	12	1/2	19 1/4
18	23 1/2	18 5/16	5/8	2 1/4	18 7/8	16	5/8	21 1/4
20	25 1/2	20 5/16	5/8	2 1/2	20 7/8	16	5/8	23 1/8
22	28 1/2	22 3/8	11/16	2 1/2	23	16	5/8	26
24	30	24 3/8	11/16	2 1/2	25	16	5/8	27 3/4

Flanges are smooth forged to above dimensions, carried in stock with and without bolt holes. All flanges shipped plain unless otherwise ordered Above standard adopted by leading manufacturers of riveted steel pipe [38]

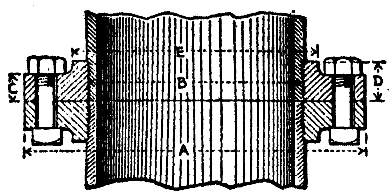


FIG. 153.—Standard companion flanges (Table 86).

Table 86. Standard Companion Flanges, Forged and Rolled Steel
(Dimensions in Inches)

Nominal size, in.	Outside diam. A	Actual bore B	Thickness C	Depth of hub D	Diam. of hub E	No. of bolts	Size of bolts	Diam. bolt circle
1	2	3	4	5	6	7	8	9
2	6	2 1/8	5/8	1	3 1/8	4	5/8	4 3/4
2 1/2	7	2 1/2	11/16	1 1/16	3 5/8	4	5/8	5 1/2
3	7 1/2	3 1/8	3/4	1 1/8	4 5/16	4	5/8	6
3 1/2	8 1/2	3 5/8	13/16	1 3/16	4 7/8	4	5/8	7
4	9	4 1/8	15/16	1 3/16	5 3/8	4	3/4	7 1/2
4 1/2	9 1/4	4 5/8	15/16	1 1/4	5 13/16	8	3/4	7 3/4
5	10	5 1/8	15/16	1 5/16	6 7/16	8	3/4	8 1/2
6	11	6 3/16	1	1 7/16	7 9/16	8	3/4	9 1/2
7	12 1/2	7 3/16	1 1/16	1 1/2	8 5/8	8	3/4	10 3/4
8	13 1/2	8 3/16	1 1/8	1 5/8	9 11/16	8	3/4	11 3/4
9	15	9 3/16	1 1/8	1 3/4	10 5/8	12	3/4	13 1/4
10	16	10 5/16	1 3/16	1 7/8	11 15/16	12	7/8	14 1/4
12	19	12 5/16	1 1/4	2 1/16	14 1/8	12	7/8	17
14	21	13 1/2	1 3/8	2 3/16	15 7/16	12	1	18 3/4

Flanges are furnished smooth forged to above dimensions, except allowance on face for finishing. Standard companion flanges, made to dimensions of A. S. M. E. standard cast-iron flanges, can be used with standard valves and fittings. They are sufficiently strong for high pressures and in many cases may be used in place of extra-heavy standard. The maker is prepared to furnish these flanges faced, drilled, and threaded, every threaded flange being tested with Brigg's Standard gage to insure perfect fit of thread.[35]

Table 86A. Weight of 100 Bolts with Square Heads and Nuts, Pounds

Length under head to point, inches	Diameter of bolts, inches												
	1/4	5/16	3/8	7/16	1/2	5/8	3/4	7/8	1	1 1/4	1 1/2	1 3/4	2
1 1/2	4.0	7.0	10.5	15.2	22.5	39.5	63.0	.....	.....	.....	.....	.....	.....
1 3/4	4.4	7.5	11.3	16.3	23.8	41.6	66.0	.....	.....	.....	.....	.....	.....
2	4.8	8.0	12.0	17.4	25.2	43.8	69.0	109.0	163	.....	.....	.....	.....
2 1/4	5.2	8.5	12.8	18.5	26.5	45.8	72.0	113.3	169	.....	.....	.....	.....
2 1/2	5.5	9.0	13.5	19.6	27.8	48.0	75.0	117.5	174	.....	.....	.....	.....
2 3/4	5.8	9.5	14.3	20.7	29.1	50.1	78.0	121.8	180	.....	.....	.....	.....
3	6.3	10.0	15.0	21.8	30.5	52.3	81.0	126.0	185	358	589	900	1312
3 1/2	7.0	11.0	16.5	24.0	33.1	56.5	87.0	134.3	196	375	613	934	1355
4	7.8	12.0	18.0	26.2	35.8	60.8	93.1	142.5	207	392	638	967	1399
4 1/2	8.5	13.0	19.5	28.4	38.4	65.0	99.1	151.0	218	409	662	1001	1442
5	9.3	14.0	21.0	30.6	41.1	69.3	105.2	159.6	229	426	687	1034	1486
5 1/2	10.0	15.0	22.5	32.8	43.7	73.5	111.3	168.0	240	443	711	1068	1529
6	10.8	16.0	24.0	35.0	46.4	77.8	117.3	176.6	251	460	736	1101	1573
6 1/2	.....	.....	25.5	37.2	49.0	82.0	123.4	185.0	262	477	760	1135	1616
7	.....	.....	27.0	39.4	51.7	86.3	129.4	193.7	273	494	785	1168	1660
7 1/2	.....	.....	28.5	41.6	54.3	90.5	135.0	202.0	284	511	809	1202	1703
8	.....	.....	30.0	43.8	59.6	94.8	141.5	210.7	295	528	834	1235	1747
9	.....	.....	.....	46.0	64.9	103.3	153.6	227.8	317	562	883	1301	1835
10	.....	.....	.....	48.2	70.2	111.8	165.7	244.8	339	596	932	1368	1922
11	.....	.....	.....	50.4	75.5	120.3	177.8	261.9	360	630	982	1435	2009
12	.....	.....	.....	52.6	80.8	128.8	189.9	278.9	382	665	1031	1502	2096
Per inch additional	1.4	2.1	3.1	4.2	5.5	8.5	12.3	16.7	21.8	34.1	49.1	66.8	87.2

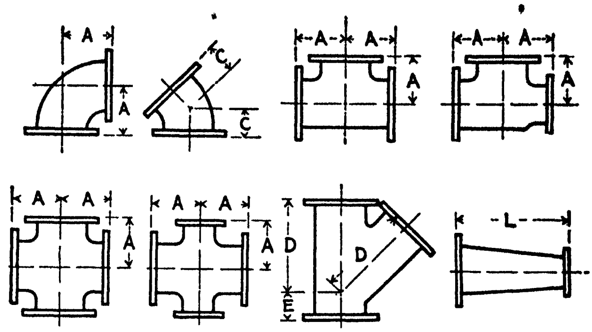


FIG. 154—Flanged fittings for pipes (Table 87) See also Table 101, p 391

Table 87 Flanged Fittings for Pipes*
(Dimensions in Inches)

Diam	Center to face A	Center to face C	Center to face D	Center to face F	Face to face L
①	②	③	④	⑤	⑥
3	$3\frac{5}{8}$	$2\frac{1}{2}$	9	$2\frac{1}{2}$	
4	$4\frac{3}{8}$	$2\frac{15}{16}$	11	$2\frac{3}{4}$	12
5	$5\frac{1}{4}$	$3\frac{1}{4}$	12	3	12
6	$6\frac{1}{4}$	$3\frac{3}{4}$	$13\frac{1}{2}$	$3\frac{1}{4}$	12
7	$7\frac{1}{4}$	$4\frac{1}{8}$	15	$4\frac{1}{4}$	12
8	$8\frac{1}{4}$	$4\frac{1}{2}$	17	5	22
9	$9\frac{1}{4}$	$5\frac{1}{16}$	$18\frac{1}{2}$	$5\frac{1}{4}$	22
10	$10\frac{1}{4}$	$5\frac{7}{16}$	21	$5\frac{1}{2}$	22
11	11	$5\frac{3}{4}$	$22\frac{1}{2}$	$5\frac{3}{4}$	22
12	$12\frac{1}{4}$	$6\frac{5}{16}$	24	6	22
13	13	$5\frac{1}{2}$	26	$6\frac{1}{4}$	22
14	14	6	27	$6\frac{1}{2}$	22
15	15	$5\frac{1}{2}$	$29\frac{1}{2}$	$6\frac{3}{4}$	22
16	16	$6\frac{11}{16}$	$31\frac{1}{2}$	7	22
18	$16\frac{1}{2}$	$8\frac{1}{2}$	35	$7\frac{1}{2}$	32
20	18	$9\frac{1}{2}$	$38\frac{1}{2}$	8	32
22	20	10	41	9	32
24	22	11	44	10	32
26	23	13			42
28	24	14			42
30	25	15			42
32	26	16			42
34	27	17			42
36	28	18			42
40	30	20			40

Center to face dimensions on other specials On reducing outlets to tees crosses and Y s—no change on increasing outlets to tees—same as respective standards on increasing outlets to crosses—determined by standard of largest opening

* Spiral Pipe Standard For 1914 standard of National Assn of Master Steam & Hot Water Fitters, see pp 391 and 392

Trade names of fittings in Fig 154 reading in normal order are one fourth bend (or elbow), one-eighth bend (or elbow), tee, reducing tee, cross, reducing cross, Y-branch reducer (or increaser)

FIG. 155.—Tank flange (Table 88).

Table 88. Forged Steel Tank Flanges
(Dimensions in Inches)

Nominal size, in.	Outside diam.	Thickness	Depth of hub	Diam. of hub
1	5	$\frac{3}{16}$	$\frac{11}{16}$	$1\frac{7}{8}$
$1\frac{1}{4}$	$5\frac{1}{2}$	$\frac{1}{4}$	$\frac{11}{16}$	$2\frac{3}{16}$
$1\frac{1}{2}$	6	$\frac{1}{4}$	$\frac{3}{4}$	$2\frac{9}{16}$
2	$6\frac{1}{2}$	$\frac{1}{4}$	$\frac{3}{4}$	$3\frac{1}{16}$
$2\frac{1}{2}$	$7\frac{1}{2}$	$\frac{5}{16}$	1	$3\frac{9}{16}$
3	8	$\frac{5}{16}$	$1\frac{1}{8}$	$4\frac{5}{16}$
$3\frac{1}{2}$	$8\frac{1}{2}$	$\frac{5}{16}$	$1\frac{1}{8}$	$4\frac{13}{16}$
4	$9\frac{1}{2}$	$\frac{3}{8}$	$1\frac{3}{16}$	$5\frac{3}{8}$
$4\frac{1}{2}$	10	$\frac{3}{8}$	$1\frac{1}{4}$	$5\frac{7}{8}$
5	11	$\frac{3}{8}$	$1\frac{5}{16}$	$6\frac{9}{16}$
6	12	$\frac{3}{8}$	$1\frac{3}{8}$	$7\frac{11}{16}$

Threaded with standard taper thread. Following are smallest circles to which tank flanges are bent: 1-in., $1\frac{1}{4}$-in., $1\frac{1}{2}$-in., 18-in. circle. 2-in., $2\frac{1}{2}$-in., 3-in-, 30-in. circle. $3\frac{1}{2}$-in., 4-in., $4\frac{1}{2}$-in., 5-in., 6-in., 48-in. circle. (Am. Spiral Pipe Works.) Flanged connections may be substituted for screwed.

Standard tank flanges are made to meet a need for flanges lighter than the standard boiler flanges, and are especially suited to thin plate work, as they are readily drawn into place.

PIPE WELDING AND CUTTING*

Use. Welding and cutting by means of oxyacetylene† torch are used extensively in shop and field for fabricating and altering pipe lines of steel and wrought iron; they are also employed to some extent on cast-iron lines. They have the merit of avoiding removal of the pipe to the shop for modifications, as commonly all operations can be completed in field. They make feasible refinements in alinement often desired by the hydraulician but impracticable with cast pipe and lead or flange joints.

Wagner‡ states, after experience in welding more than 150 mi. of high-pressure oil lines, that three essentials for success are good equipment, competent operators, right welding rod (metal for making the joint).

Welders must be carefully selected by test. In the test found most satisfactory, each candidate makes sample welds of short pieces of pipe, which are cut into strips and examined for penetration and to discover whether the welder made "icicles." The pieces are then put into a vise and bent until fractured. With this test it is easy to find out whether a welder gets fusion and not merely adhesion. Each welder should stamp an identifying mark on the pipe near each joint that he welds.[36]

Specials. Flame-cutting tools combined with welding outfits make possible the cutting on the job of filling-in pieces and outlets for "specials" or other connections, at any angle. Specials, even the most complicated, can often be built upon the job from straight pipe and other simple parts. Space as well as time and expense can be saved. Flanges and gaskets can be avoided,

* See "Some Management Problems on Pipe Welding Jobs," by G. O. Carter, Consulting Engineer, Linde Air Products Co., *J. Am. Welding Soc.*, Dec., 1923, p. 35.
† Electric welding is also extensively used.
‡ See second foot-note, p. 354.

or, if flanges have been used at certain places, they can be sealed at their edges and gaskets omitted. Alterations and extensions are facilitated.

Cast-Iron Pipe. As a rule, it does not pay to weld cast-iron pipe and fittings because of the expensive precautions necessary. An example of exceptions is a broken large special casting where an extensive shutdown has to be avoided. A cracked 48-in. cast-iron main at Baltimore was repaired in place.[56] See also "The Use of Welding for Pipe Lines," by Sforzini, *Power*, Nov. 20, 1923, p. 798. Many water departments are now equipped for welding; see Bloomington, Ill., experience in *Am. City*, 1924, p. 635.

Steel Pipe. The many advantages of modern welding and cutting, in extension and repairs, as well as new construction, give increasing preference to steel pipe, extending to sizes too small to be riveted. For useful information and tables on oxyacetylene pipe welding and cutting, see "Gas Welded Pipe Joints" (The Linde Air Products Co.), New York, 1923.

Failures of Large Sizes. Of an installation of four welded pipes two failed at the weld, one after being in service 7 years. Rupture showed the weld blistered nearly the entire length, and for 9 in. a flaw two-thirds through the metal. At the time of break, very cold water was being drawn; it was supposed that contraction caused extra stress at the weld (pipe was flattened at joint). Numerous lengths of tested pipe have had to be discarded; a test of a few minutes is not sufficient. It is advisable to have as few cross connections as possible, and these, where feasible, normally closed. At another plant, of two lap-welded pipes, one opened at a weld shortly after installation; after 7 years another section opened at the weld; inspection showed an imperfect weld and shearing of the outer portion of the steel from the inner along the neutral axis. At a third plant of two lines, partly 30-in., five sections were rejected in the field for defective welds; examination by drilling showed in cases less than half of the metal in contact. In another case, 24-in. pipe for a maximum head of 1068 ft. tested at shop, one section was found to have opened 5 ft. along the weld; after being in service 2 weeks another section opened at the weld.[46a] These failures were recorded 20 yr. ago, before the art of welding had advanced.

Progress in Welding. The art has advanced rapidly in recent years. The Hydraulic Power Committee mentions[23] "great improvement in manufacturing methods insuring a very uniform product."

Strength of Weld. Hydraulic Power Committee[23] reports only one failure of recently welded pipe, and this one indicated no flaw in the weld itself.

It is universal practice to consider the efficiency of the weld as 90 per cent. This is the value guaranteed by the manufacturers; but innumerable tests show that the weld is practically as strong as the plate. Tests also show that the elastic limit of the steel is altered very little by welding, so that it is proper to assume the elastic limit of the welded joint as 90 per cent. of that of the original plate where the factor of safety is based on the elastic limit.

Continental Iron Works reports an 18-in. steam main bursting under 1750 lb. test pressure, with the split 4 in. from the seam.

Tests, at University of Illinois, of oxyacetylene-flame-welded joints in steel plate up to 1 in. thick showed joint efficiency for static tension from 72 to 128 per cent.; for tensile impact, 32 to 97 per cent. (*Bull.* 98, Eng. Exp. Sta., 1917). Tests at University of Kansas showed strength of a welded connection equal to that of unwelded pipe.[37]

WELDED PIPE

Manufacture. Commercial sizes of welded pipe, which are those commonly used for water-service pipes and heating systems, are made by pressing together the heated plates in rolls. Pipes from ⅛ to 3 in. are shaped and butt welded in one operation; sizes above 3 in. are lap welded, the welding being accomplished in rolls exerting a heavy pressure. The Continental Iron Works, N. W. Kellogg Co., and others make large pipe by heating the pipe to welding temperature, and rolling between two rolls, one of which is shoved against the other by means to exert a heavy pressure. Welding up to 2-in. plate is feasible.

The joining of two pieces of metal by heating two edges in contact to a fusing temperature was known as autogenous welding, but "fusion welding" is preferred term. When the weldable edges are subjected to hammering or pressure, the process is known as forge welding. The Hydraulic Power Committee advocates that penstock pipe be forge welded; fusion welding is not considered reliable enough for this class of work.[23]

Hammer welding consists of heating the lapped edges of the pipe to weldable temperature, and then subjecting them to the repeated blows of a drop hammer. Hammer welding of large steel pipes originated in Germany. Continental Iron Works, Brooklyn, have made large tubes for years. Other American firms equipped for hammer welding are: the American Welding Co., Carbondale, Pa.; American Spiral Pipe Works, Chicago, Ill.; Blaw-Knox Co., Pittsburg, Pa., and the National Tube Co., Pittsburg, Pa. The last company has recently put in equipment to make 20- to 96-in. pipe, with wall thickness of ¼ to 1½ in. Plates used by National Tube Co. conform to Specification A 78–21 T of A. S. T. M.

Advantages. In penstock design, it has been found in recent years that welded pipe is more economical than riveted steel.[23] For water-supply systems welded pipe, with some type of calked joint (see "Matheson" and "Converse," p. 352), reduces weight 75 per cent. for the same pressure and saves in joints per mi. (in an 8-in. line, 3940 lb. of lead per mi.). For service pipes, the smaller sizes of welded pipe, galvanized, have gradually taken precedence on account of lower price. Corrosion is a serious consideration; see Gerhard's studies, *J. A. S. M. E.*, Vol. 40, 1918, p. 945.

Disadvantages. The mill scale usually formed on rolled steel or iron has been found from tests to be a potential source of pitting. Commercial pipes having a particularly heavy and tightly adhering scale are susceptible to pitting. The National Tube Co. has recently developed a process for producing a pipe free from welding scale. Welded pipe is heavier than the same class of riveted steel pipe; this added thickness is held by the manufacturers to constitute an additional factor against corrosion.

Field Joints. See p. 352.

COMMERCIAL IRON AND STEEL PIPE

Nomenclature. "Steel Pipe," "Wrought Steel," and "Wrought Pipe" are trade names applied to welded pipe made of steel. Gerhard[52] protests that the last two names convey a wrong impression. Pipe made of wrought iron is known by trade names, such as "Byers" or "Reading" and as "genuine" or "guaranteed wrought-iron." A large percentage of pipe now used in water-supply services is welded steel. Gerhard's investigation of actual installations[52] disclosed the superiority of wrought iron in services.

Designations. Standard pipe, known as Briggs standard, is designated by its nominal inside diam.; this is not the actual diam. (*cf.* Table 89). "Extra heavy" and "double extra heavy" have the same outside diam. as "standard" pipe, thickening of shell being secured by diminution of waterway; they are designated by the diam. of the "standard" sections. Calculations should be guided accordingly. Above 12 in., pipe is known as O.D., and is designated by its outside diam.

To Distinguish Wrought-iron from Steel Pipe. Many tests are described in *E. C.*, Oct. 13, 1915, p. 295. W. R. Conard advocates immersing the metal specimen in a bath of 1 part sulphuric acid and 3 parts water, for 8 hr. The pitting of the metal will show a granular structure if steel, a fibrous structure if of wrought iron.*

Standard Specifications. A. S. T. M.: Welded and Seamless Steel Pipe, Serial designation A 53-21, adopted 1915, revised 1918, 1921; and tentative Specifications for Steel Plates for Forge Welding, Serial designation A 78-21 T, issued, 1919, revised 1920, 1921. The American Bureau of Welding and also Comm. 12, A. W. W. A., are preparing specifications (1925).†

Specifications‡ for Service Pipes. Galvanized-iron pipe shall be standard size, guaranteed wrought-iron pipe, galvanized, full weight, equivalent to pipe manufactured by A. M. Byers & Co., Pittsburgh. All pipe above 1¼-in. internal diam. shall be lap welded. All pipe less than and including 1¼-in. inside diam. may be butt welded. Weights shall not vary more than 5 per cent. from following:

Inside diam., in.	Weight per foot, lb.	Inside diam., in.	Weight per foot, lb.
½	0.84	2½	5.77
¾	1.12	3	7.54
1	1.67	3½	9.05
2	3.66	4	10.72

Connections shall be made to main pipe by means of standard water-pipe clamp with threaded outlet. When possible, connections shall be made to main line at a tapped plug. All threads of screw connections shall be unbroken and cut full depth, and, before connections are made, well covered with steam-fitter's cement. The pipe shall be laid with a cover of not less than 2 ft. and tested by hydrostatic pressure to 300 lb. per sq. in.

* See also *E. R.*, July 22, 1916, p. 110.
† The A. S. T. M. adopted specifications for wrought-iron pipe up to 6-in. diam. in 1918.
‡ Seattle, Wash., Standards, 1910.

Table 89. Steel or Wrought-iron Pipe, Standard Dimensions and Weights

See also Page 305

Nominal inside diam., in.	Actual inside diam., in.	Actual* outside diam., in.	Nominal thickness, in.	Internal circumference, in.	External circumference, in.	Length of† pipe per sq. ft. of inside surface, ft.	Length† of pipe per sq. ft. of outside surface, ft.
$\frac{1}{8}$	0.270	0.405	0.068	0.848	1.272	14.15	9.44
$\frac{1}{4}$	0.364	0.54	0.088	1.144	1.696	10.50	7.075
$\frac{3}{8}$	0.494	0.675	0.091	1.552	2.121	7.67	5.657
$\frac{1}{2}$	0.623	0.84	0.109	1.957	2.652	6.13	4.502
$\frac{3}{4}$	0.824	1.05	0.113	2.589	3.299	4.635	3.637
1	1.048	1.315	0.134	3.292	4.134	3.679	2.903
$1\frac{1}{4}$	1.380	1.66	0.140	4.335	5.215	2.768	2.301
$1\frac{1}{2}$	1.611	1.9	0.145	5.061	5.969	2.371	2.01
2	2.067	2.375	0.154	6.494	7.461	1.848	1.611
$2\frac{1}{2}$	2.468	2.875	0.204	7.754	9.032	1.547	1.328
3	3.067	3.5	0.217	9.636	10.996	1.245	1.091
$3\frac{1}{2}$	3.548	4.0	0.226	11.146	12.566	1.077	0.955
4	4.026	4.5	0.237	12.648	14.137	0.949	0.849
$4\frac{1}{2}$	4.508	5.0	0.246	14.153	15.708	0.848	0.765
5	5.045	5.563	0.259	15.849	17.475	0.757	0.629
6	6.065	6.625	0.280	19.054	20.813	0.63	0.577
7	7.023	7.625	0.301	22.063	23.954	0.544	0.505
8	7.982	8.625	0.322	25.076	27.096	0.478	0.444
9	9.001	9.688	0.344	28.277	30.433	0.425	0.394
10	10.019	10.75	0.366	31.475	33.772	0.381	0.355
11	11.0	11.75	0.375	34.55	36.91	0.34	0.32
12	12.0	12.75	0.375	37.70	40.05	0.32	0.30

* Useful for figuring clearance.
† Useful for figuring heat losses in power plants, heating surface of boiler tubes, etc.

Table 89. Steel or Wrought-iron Pipe, Standard Dimensions and Weights.— *(Continued)*

See also Page 305

Nominal inside diam., in.	Internal area, sq. in.	External area, sq. in.	Length of pipe containing 1 cu. ft., feet	Nominal* weight per ft., lbs.	No. of threads per in. of screw	Contents in gal. per ft.	Size drills, in., to be used for tapping
$\frac{1}{8}$	0.0572	0.129	2500.0	0.243	27	0.0006	$\frac{21}{64}$
$\frac{1}{4}$	0.1041	0.229	1385.0	0.422	18	0.0026	$\frac{27}{64}$
$\frac{3}{8}$	0.1916	0.358	751.5	0.561	18	0.0057	$\frac{19}{32}$
$\frac{1}{2}$	0.3048	0.554	472.4	0.845	14	0.0102	$\frac{23}{32}$
$\frac{3}{4}$	0.5333	0.866	270.0	1.126	14	0.0230	$\frac{15}{16}$
1	0.8627	1.357	166.9	1.67	$11\frac{1}{2}$	0.0408	$1\frac{3}{16}$
$1\frac{1}{4}$	1.496	2.164	96.25	2.258	$11\frac{1}{2}$	0.0638	$1\frac{15}{32}$
$1\frac{1}{2}$	2.038	2.835	70.65	2.694	$11\frac{1}{2}$	0.0918	$1\frac{23}{32}$
2	3.355	4.430	42.36	3.667	$11\frac{1}{2}$	0.163	$2\frac{3}{16}$
$2\frac{1}{2}$	4.783	6.491	30.11	5.773	8	0.255	$2\frac{11}{16}$
3	7.388	9.621	19.49	7.547	8	0.367	$3\frac{5}{16}$
$3\frac{1}{2}$	9.887	12.566	14.56	9.055	8	0.500	$3\frac{13}{16}$
4	12.730	15.904	11.31	10.728	8	0.653	
$4\frac{1}{2}$	15.939	19.635	9.03	12.492	8	0.826	
5	19.990	24.299	7.20	14.564	8	1.020	
6	28.889	34.471	4.98	18.767	8	1.469	
7	38.737	45.663	3.72	23.410	8	2.00	
8	50.039	58.426	2.88	28.348	8	2.61	
9	63.633	73.715	2.26	34.077	8	3.30	
10	78.838	90.762	1.80	40.641	8	4.08	
11	95.03	108.43	1.50	45.0	8	4.93	
12	113.09	127.67	1.27	48.98	8	5.87	

Table 90. Dimensions and Weights of Large Outside Diameter Wrought-iron Pipe[35] *

Thickness, in.	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
Outside diam. of pipe, in.	Weight per ft., lbs.	Weight per ft., lbs.	Weight per ft., lbs.	Weight per ft., lbs.	Weight per ft., lbs.	Weight per ft., lbs.	Weight per ft., lbs.	Weight per ft., lbs.	Weight per ft., lbs.
14	36.75	45.72	54.61	63.42	72.16	80.80	89.36	97.84	106.2
15	39.42	49.06	58.62	68.10	77.50	86.81	96.03	105.2	114.2
16	42.09	52.40	62.63	72.78	82.85	92.83	102.7	112.5	122.2
18	47.44	59.08	70.65	82.14	93.54	104.8	116.1	127.2	138.3
20	52.78	65.76	78.67	91.49	104.2	116.9	129.4	141.9	154.3
22	......	72.44	86.68	100.8	114.9	128.9	142.8	156.6	170.3
24	......	79.13	94.70	110.2	125.6	140.9	156.2	171.3	186.3
26	......	......	102.7	119.5	136.3	152.9	169.5	186.0	202.4
28	......	......	110.7	128.9	147.0	165.0	182.9	200.7	218.4
30	......	......	....	138.2	157.7	177.0	196.3	215.4	234.4

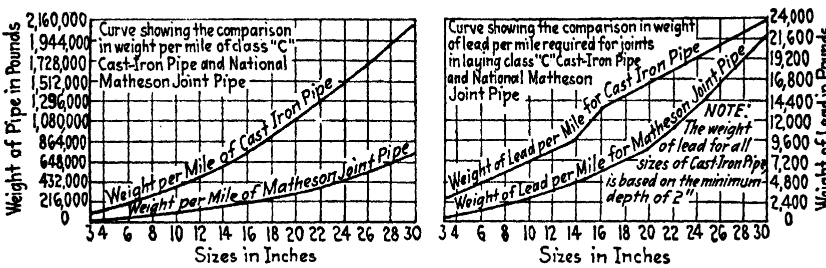
* Add 2 per cent. for weight of steel pipes.

Table 91. Lap-welded Artesian-well Casing

Nominal inside diam., in.	Actual outside diam., in., not including couplings	Nominal weight per ft., lbs.	No. of threads per in. of screws
2	2 1/4	2.22	14
2 1/4	2 1/2	2.82	14
2 1/2	2 3/4	3.13	14
2 3/4	3	3.45	14
3	3 1/4	4.10	14
3 1/4	3 1/2	4.45	14
3 1/2	3 3/4	4.78	14
3 3/4	4	5.56	14
4	4 1/4	6.00	14
4 1/4	4 1/2	6.36	14
4 1/4	4 1/2	9.38	14
4 1/2	4 3/4	6.73	14
4 1/2	4 3/4	9.39	14
4 3/4	5	7.80	14
5	5 1/4	8.20	14
5	5 1/4	9.86	14
5	5 1/4	12.80	11 1/2
5	5 1/4	15.88	11 1/2
5 3/16	5 1/2	8.62	14
5 3/16	5 1/2	12.49	11 1/2
5 5/8	6	10.46	14
5 5/8	6	12.04	11 1/2
5 5/8	6	14.20	11 1/2
5 5/8	6	16.70	11 1/2
6 1/4	6 5/8	11.58	14
6 1/4	6 5/8	13.32	14 & 11 1/2
6 1/4	6 5/8	17.02	11 1/2
6 5/8	7	12.34	14
6 5/8	7	17.51	11 1/2 & 10
7 1/4	7 5/8	13.55	14
7 5/8	8	15.41	11 1/2
7 5/8	8	20.17	11 1/2
8 1/4	8 5/8	16.07	11 1/2
8 1/4	8 5/8	20.10	11 1/2
8 1/4	8 5/8	24.38	11 1/2 & 8
8 5/8	9	17.60	11 1/2
9 5/8	10	21.90	11 1/2
10 5/8	11	26.72	11 1/2
11 5/8	12	30.35	11 1/2
12 1/2	13	33.78	11 1/2

Bends in welded pipes in small sizes—below 6 in.—may be made in the field cold or in the shop by heating. Pipe-bending machines are made by American Pipe Bending Machine Co., Boston, and Federal Pipe Bending Machine Co., Bayonne, among others.

Field Joints. *Couplings.* The smaller sizes of pipe, such as used for house service pipes, and up to 15-in. diam. are connected by standard couplings, furnished by the pipe companies. If there is no room for coupling, "flush joint tubing" may be used. Pipe threads are always Briggs Standard.



Comparative weight of pipe per mile.* Comparative weight of lead per mile.

FIG. 156.—National Matheson joint pipe vs. cast-iron pipe.

On large work (for oil lines, etc.) joints are screwed up by machine. On a job in Chili[39] 26 men with a machine laid 60 lengths per day, average, as against 25 lengths by 19 men with chain tongs; rough country. At stream crossings and wherever else the joint must be protected from stress which might impair its tightness, it is surrounded by a sleeve known as "River Sleeve" or "River Clamp." Among the firms handling accessories for wrought pipe lines are Oil Well Supply Co., New York. Screw pipe is most susceptible to electrolysis, as joints offer no resistance.

Calked Joints. The Converse joint is a sleeve with a bell at each end for lead filler. The Matheson joint is a bell flared onto the pipe when it is welded; it has not the depth of a cast-iron pipe lead space, and lead is saved. As made by the National Tube Co., Matheson pipe is equivalent in strength to the Class C pipe of A. W. W. specifications. It has the added advantages that it comes in 18- to 20-ft. lengths, and that there is no breakage. The shallower lead space increases possibility of leakage.

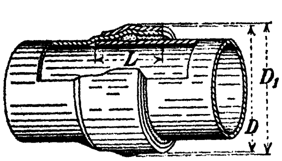


FIG. 157.—"National" Matheson joint. Sectional view.

Welded field joints[40] have been used on conduits of 22-, 24-, and 30-in. riveted pipe ($\frac{3}{16}$-in. plate) by Spring Valley Water Co., San Francisco, and on 12- to 20-in. sheet-iron pipes of East Bay Water Co.[15, 60] They figured cheaper than the riveted joint or the lead joint (see Fig. 157). All welding was done by oxyacetylene apparatus, in accordance with standard welding practice; $\frac{3}{16}$-in. soft Swedish iron rods were used. The ends of all joints were kept apart to allow for expansion. Welds were twice the plate thickness, and were carried along the longitudinal seam to a point

* National Pipe Co.

opposite the first rivet. This seam was then brazed to the second rivet, using a very soft brazing wire. Brazing was done at a much lower temperature than the welding, producing a tight seam without warping or introducing strains. If the brass cracks in cooling, it can be calked in place. A hole must be excavated on each side of the joint so that a man can lie on his right side to weld the bottom. Only one joint pulled apart; this was due to settling ground. Welded joints produce a very flexible job, as pipe can be readily modified in the field to pass any obstacles uncovered. Butte Water Co.[14] uses lap-welded

Table 92. "National" Matheson Joint Pipe
(All Weights and Dimensions are Nominal)

External diameter, inches	Thickness, inches	Max.* O. D. at end of pipe D, inches	Max.* O. D. over bell D_1, inches	Length of joint L, inches	Weight per foot, lb.: Plain ends	Weight per foot, lb.: Complete: Dipped only, or galvanized and dipped	Weight per foot, lb.: Complete: "National" coated single wrapped	Weight per foot, lb.: Complete: "National" coated double wrapped	Weight of lead per joint, pounds	Mill test, pounds per square inch
2.00	0.095	2.78	2.90	2.16	1.93	1.95	2.30	2.40	1.00	700
3.00	0.109	3.87	4.00	2.26	3.36	3.39	3.90	4.10	1.75	700
4.00	0.128	4.99	5.13	2.32	5.29	5.34	6.00	6.30	2.75	600
5.00	0.134	6.03	6.17	2.38	6.96	7.03	7.90	8.20	3.50	600
6.00	0.140	7.10	7.25	2.50	8.76	8.85	9.90	10.30	4.75	600
7.00	0.149	8.15	8.31	2.58	10.90	11.02	12.30	12.70	5.50	600
8.00	0.158	9.21	9.38	2.73	13.23	13.38	14.80	15.30	6.75	600
8.00	0.185	9.32	9.49	2.78	15.44	15.62	17.10	17.60	6.75	700
9.00	0.167	10.26	10.43	2.73	15.75	15.93	17.60	18.10	8.25	500
9.00	0.196	10.38	10.55	2.90	18.42	18.65	20.30	20.90	8.50	600
9.00	0.250	10.60	10.77	3.07	23.36	23.66	25.30	25.90	9.00	700
10.00	0.175	11.32	11.50	2.82	18.36	18.57	20.40	21.00	9.50	500
10.00	0.208	11.45	11.63	2.85	21.75	22.00	23.80	24.50	9.75	600
10.00	0.270	11.70	11.88	3.06	28.05	28.41	30.20	30.90	10.00	700
11.00	0.185	12.38	12.57	2.91	21.36	21.62	23.60	24.30	11.00	500
11.00	0.220	12.52	12.71	2.93	25.32	25.64	27.60	28.30	11.00	600
11.00	0.290	12.80	12.99	3.17	33.17	33.60	35.60	36.30	12.50	700
12.00	0.194	13.43	13.62	3.00	24.46	24.77	27.00	27.70	13.25	500
12.00	0.244	13.63	13.82	3.40	30.63	31.07	33.20	34.00	14.25	600
12.00	0.310	13.90	14.09	3.76	38.70	39.30	41.50	42.30	16.50	700
13.00	0.202	14.49	14.68	3.07	27.61	27.97	30.30	31.20	15.25	500
13.00	0.247	14.67	14.86	3.40	33.64	34.11	36.50	37.30	15.50	600
13.00	0.310	14.92	15.12	3.76	42.01	42.67	45.00	45.90	18.00	650
14.00	0.210	15.54	15.75	3.15	30.93	31.34	33.90	35.00	17.25	500
14.00	0.250	15.70	15.91	3.53	36.71	37.26	39.80	41.00	19.25	550
14.00	0.310	15.94	16.15	3.84	45.32	46.06	48.60	49.80	20.75	600
15.00	0.222	16.58	16.79	3.24	35.04	35.51	38.20	39.50	19.25	500
15.00	0.260	16.74	16.95	3.53	40.93	41.53	44.30	45.50	20.25	550
15.00	0.320	16.98	17.19	3.84	50.17	50.98	53.70	54.90	22.25	600
16.00	0.234	17.65	17.87	3.32	39.40	39.96	42.90	44.20	22.00	500
16.00	0.270	17.80	18.02	3.62	45.36	46.05	49.00	50.30	23.25	550
16.00	0.330	18.04	18.26	3.75	55.23	56.09	59.00	60.30	24.25	600
17.00	0.240	18.70	18.93	3.41	42.96	43.57	46.70	48.00	23.75	450
18.00	0.245	19.72	19.96	3.50	46.46	47.14	50.40	51.80	25.75	450
18.00	0.310	19.98	20.22	3.87	58.57	59.51	62.80	64.20	28.50	500
19.00	0.259	20.79	21.03	3.57	51.84	52.62	56.10	57.60	29.00	450
20.00	0.272	21.84	22.09	3.64	57.31	58.18	61.90	63.40	31.00	450
20.00	0.375	22.26	22.51	4.17	78.60	79.94	83.60	85.10	35.50	500
22.00	0.301	23.98	24.26	4.06	69.75	70.93	75.00	76.60	40.25	450
22.00	0.400	24.38	24.66	4.65	92.27	94.03	98.10	99.70	45.50	500
24.00	0.330	26.14	26.42	4.26	83.42	84.89	89.30	91.10	48.00	450
26.00	0.362	28.28	28.58	4.40	99.12	100.91	105.70	107.90	55.25	450
28.00	0.396	30.46	30.76	4.58	116.74	118.97	124.10	126.40	65.00	450
30.00	0.432	32.62	32.94	4.75	136.42	139.06	144.60	147.00	75.00	450

Permissible variation in weight of pipe without coating is 5 per cent. either way from standard. Shipped in random lengths. Weight per foot complete is based on a length of 18 ft. of pipe, but shipping lengths of sizes 22 in. O. D. and larger may average less than 18 ft. Columns marked "weight complete" include ring but not lead.

* See Fig. 157.

steel pipe with welded end joints where the head exceeds 300 ft. It has been suggested that welded joints reduce electrolytic action and corrosion * Fusion welding of joints in gas, steam, and oil lines has apparenty been successful On oil lines N E Wagner holds it superior to screwed couplings in taking the stresses due to changes of alinement †

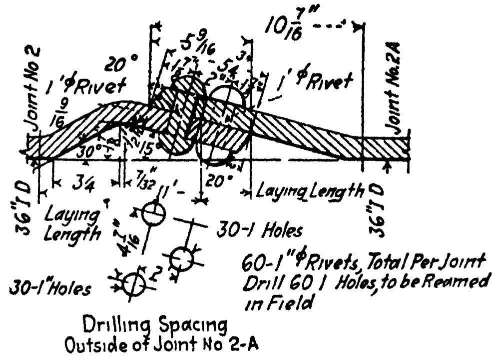


FIG 158 —Typical double riveted bump joint [38]

As many joints as practicable should be welded under conditions which permit rolling the pipe so as to keep the work approximately horizontal Necessary "bell-hole" welds can however be successfully made by skilful welders

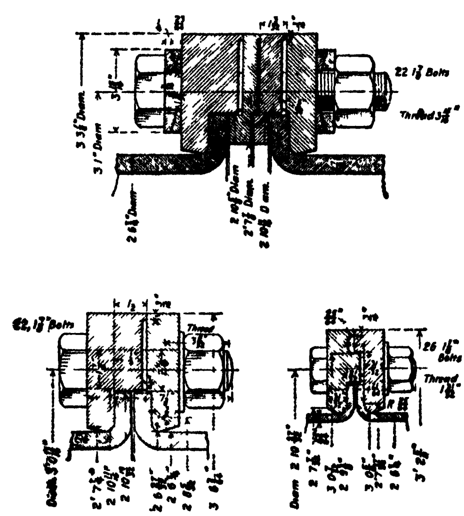


FIG. 159 —Joints for high pressure steel pipe

Penetration of fused metal to the inside of the pipe, making obstruction to scrapers used in pipe cleaning, and causing other troubles, may be avoided by holding the torch so that the flame is parallel to the plane of welding.

* American Rolling Mill Co , Middletown, Ohio

† Paper before International Acetylene Assn , October, 1922 See also "Applications and Tests of Large Diameter Electric Welded Pipe," Nead and Kenyon, *J Am Welding Soc* , April, 1922.

Flange Joint. Types of flanges are shown in Figs. 151 to 155. Flanges may be screwed, welded, riveted, or peened on.* Screwed flanges are most common between 1¼ and 15 in. Gaskets are required, see p. 392.

Bump Joint (Fig. 158). On penstocks, for all pipes 26 in. min. diam. and above, the common type of "bump joint" is preferred. This joint is of special value in laying, as it permits slight deviations in alinement in order to maintain the proper line. Bump joints can be either double or single riveted, but double riveting is preferred [23] Rivet holes should be drilled 1/16 in. small in the shop and reamed to size in the field after the pipe is in place. As it is not possible to rivet bump joints in pipes smaller than 26 in., below this diam. flange joints must be used.[38] The advantage of the bump joint over the in-an-out riveted joint or the tapered joint lies in the better waterway afforded.

Joints for High-pressure Steel Pipes.[46b] For Necaxa, Mex., power plant, 30-in. pipes to carry water under 1450-ft. head were forged complete with flanges from one sheet of steel for each 30-ft. section. Longitudinal seams were lap-welded. Pipes were tested in shop to 1¾ times maximum working head. Maximum velocity of water was 18 ft. Special joints used are shown in Fig. 159. Pipes were made by Actien Gesellschaft Ferrum, Kottowitz, Germany. Rubber gaskets were used.

Mannesmann seamless steel tubes,[43] made at Swansea, England, under German patents, were used in 1910 for distributing mains at Victoria, B. C. Tubes came in 28-ft. lengths; coated with asphalt, wrapped with jute, and again coated with asphalt. Prices per ft. paid by city for pipe delivered along trench: 12-in., $1.59; 8-in., 72 cts.; 6-in., 52 cts.; 4-in., 29½ cts. Specials, 21 cts. per lb. Pressure not given.

Installations. Continental Iron Works, Brooklyn, fabricated two steel pipe lines: 1100 ft. of 37-in., Guanajuata Power & Electric Co., Zamora, Mex., thickness, ⅜ to 7/16 in ; 4790 ft. of 43-in. pipe, 1⅛ in. thick, Yukon Consolidated Gold Field Co., 500 lb. working pressures. On the Yukon, joints were bell and spigot, tapered, and drilled for rivets after being put together. Bends, up to 4°, were made by beveling plates of adjacent sections, and welding. Two makes,[44] one American and one German, were used; latter was not entirely satisfactory. One section cracked along a weld for 10 ft. on being dropped into trench; this was patched and reinforced, and gave no more trouble. Another section burst under 835-ft. test head, due to flaw in metal. Forge-welded steel-plate pipes, in lengths up to 50 or 60 ft., have been made in Germany for a number of years and used throughout Europe and in more distant parts of world, for water supply and power. Sixteen- and eighteen-inch Matheson pipe was used on 24-mi. conduit for Oregon City. Manufacturers and Am. Welding Society can furnish data regarding many installations. On Moccasin Creek penstock, Hetch-Hetchy, the 54-in. hammer-welded pipe is under 1315-ft. head [53] Field joints are riveted. A 30-in. hammer-welded pipe with riveted joints was used in Spavinaw conduit, Tulsa.[59] American Spiral Pipe Works installed a 48-in. line for Denver Union Water Co.; 16-, 28-, and 36-in. lines for Aberdeen, Wash.†

* "Byers" pipe can now be Van Stoned.
† See also description of 24- and 26-in. Butte pipe in *E. N. R.*, Nov. 25, 1920, p. 1032.

EXPERIENCE*

Leakage. General experience with steel mains is that they tend to grow tighter with age, unless perforations occur. A 48-in. steel main in Philadelphia, after being tested and recalked, had leakage of 7000 to 10,000 gal. per mi. per 24 hr. under pressure of 160 lb. per sq. in. A 72-in. steel main 10.8 mi. long, tested in sections, averaged 0.059 cu. ft. per sec. per mi. leakage = 38,000 gal. per mi. per 24 hr.; pressure 0 to 59 lb. Tests on some Brooklyn conduits resulted as follows: (1) 66-in. pipe, 3.2 mi. long, 33 per cent. riveted steel, remainder of lock-bar type; field joints every 30 ft.; leakage under 65 to 95 lb. test pressure was 12,300 gal. per mi. per day (3.91 gal. per lin. ft. of field joint per 24 hr.). (2) 66-in. pipe, 30 per cent. riveted steel, remainder of lock-bar type; field joints every 30 ft. Leakage under pressure of 48 to 117 lb. per sq. in. was 9800 gal. per day per mi. (3.10 gal. per lin. ft. of field joints per 24 hr. See also p. 428. On tests on 36-in. Sooke River conduit, leaks at 200 lb. pressure held for 20 min. were calked until loss did not exceed 1.2 gal. per 24 hr. per ft. of pipe.[47] A 36-in. pipe at Schenectady was backfilled and stood empty for 2 years. When water was turned in, leaks developed at circumferential joints; this was remedied by welding all joints inside and outside.[48] A 48-in. pipe in Vancouver, B. C., German built, was repaired by heavy butt straps around the leaky circumferential welded joints.[49]

Temperature Effect. Steel pipe should be anchored against temperature movements. Never leave a long length uncovered in winter between two lengths anchored, or the rivets in the circular seams of the exposed portion will be subjected to shear by temperature changes. On a 72-in. steel main in Brooklyn, 1909, the lead was pulled 2 in. out of joints 9 in. deep, at a valve, at night, by temperature effect on 3500 ft. of exposed line on either side, and forced back partly by day. Temperature ranged from about 38 to 60°F. On tangents, usually the only force to be resisted by the circular seams is that due to temperature. Assuming a temperature variation of 45°, modulus of elasticity for steel, 30,000,000; tensile strength, 55,000 lb. per sq. in.; elongation per deg., $1 \div 148{,}000$, and factor of safety of 3 necessitates a joint having $(30{,}000{,}000 \times 45) \div (148{,}000 \times 55{,}000 \div 3) = 50$ per cent. efficiency. These temperature stresses cause shear in the rivets of field seams which generally determines the spacing of the rivets on single-riveted circular seams. The 48-in. line for East Jersey Water Co. stood empty the first winter and thermometrical records showed that the temperature went below 32°F. on several occasions. The second winter the temperature conditions were worse, as it was tested under water pressure, and the pipe was repeatedly filled and emptied. The temperature of both incoming air and incoming water was around 32°. Portions stood empty or full of still water in severely cold weather. "No effect has ever been observed as resulting from this treatment of the pipe line."[1]

Temperature movements of 7 in. in 3078 ft. were observed on Catskill aqueduct siphons, for 30°F. temperature change.[25]

Steel-pipe Deformation. Kuichling[50] has seen earth fill 6 to 8 ft. deep reduce 10 per cent. the vertical diam. of 36- to 72-in. steel pipes, ¼ to ½ in.

* For durability, see p. 325.

thick. He concludes that ordinarily 5 or 6 ft. backfill will produce stresses near elastic limit. Stiffening rings of steel or concrete should be used on deep pipes. Thoroughly tamp the backfill up to the horizontal diam. The 6-in. concrete jackets of the Catskill steel pipes limit deformation. As delivered, the 11-ft. 3-in. steel pipe had a vertical diam. of 10.8 ft. When filled with water, this flattened to 10.1 ft. with $\frac{7}{16}$-in. plates, and 10.35 ft. with $\frac{5}{8}$-in. plates. Under 150-ft. head, the vertical diam. had become 10.95 ft.

A "cross-sectioner" used to measure out-of-roundness of large steel pipes on Catskill aqueduct is described in *E. N.*, Mar. 26, 1914, p. 680. A 42-in. steel pipe, Portland, Ore., was flattened 4 in. by careless backfilling, causing uneven distribution of load. Tests showed no leakage under distortion of $8\frac{3}{4}$ in., although shortening of only $1\frac{5}{8}$ in. caused permanent set of $\frac{1}{2}$ in.[51]

A 66-in. steel pipe ($\frac{5}{16}$-in. plate) in a tunnel in Pennsylvania collapsed in 1912, due to sudden withdrawal of water, so that vertical diam. became 6 in. Pipe was restored to normal shape in 2 hr. by gradually filling with water under pressure. Several hundred feet were involved. Cost of re-calking was $40.

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CHAPTER 16

WOOD-STAVE PIPE*

Essentials. Wood-stave pipe is made of staves of selected wood shaped by machine, placed side by side and end to end, and held to position and tightness by metal bands clamped around the outside. The bands take the water-pressure load, and, as they will stretch under this load, they are put in initial tension when the pipe is made, so that elongation in service will not cause leakage. Staves should have thickness enough to assure (*a*) transmission of the water-load to the bands; (*b*) durability, (*c*) crushing strength sufficient to prevent the bands sinking in when tightened. Thin staves (not over $1\frac{1}{2}$ in.) are desirable to assure the complete saturation which makes for long life; thicker staves are required for strength and rigidity in very large pipes. Watertightness between edges of staves depends on initial compression produced by the bands. *Continuous-stave* pipe is built up in the field from long staves, the ends of which are made watertight by metal plates in saw kerfs (see p. 360). *Factory-made* or *machine-made* pipe is manufactured in the factory, metal spirals of wire or bands being wrapped on continuously by machine, the pipe being joined in the trench by collars or mortise-and-tenon joints (see p. 364).

Merits. Wood pipe was in use in 1922 in at least 500 water undertakings in the United States; some had been in service 46 years; at least 25 per cent. had given "good satisfaction."[1] Hatton[2a] reported in 1907 that stave pipe of good soft white pine, Douglas fir, cypress, or even red cedar, well selected, free from dry or black knots, either air or kiln dried, free from cracks, well jointed and secured with steel bands protected from corrosion, will last, under average conditions, as long as cast-iron pipe. Authorities differ (see p. 369). Where properly designed and constructed, Ledoux[3] concludes that wood pipe has been satisfactory for pressures below 150 lb. (320-ft. head). Low first cost† and high carrying capacity at all ages must be balanced against shorter life and greater leakage‡ than for pipes of other materials. Assuming first cost of wood pipe as unity, Taylor[4] estimated for a 30-in. diam. pipe at Norfolk, Va., in 1922, reinforced concrete would cost 1.7 and cast iron, 2.4. Correcting these for relative leakage, durability, and capacity, the figures become, respectively 1.6, 1.7, and 3.4.§

The foregoing figures do not evaluate: (*a*) freedom from expansion, electrolysis, and damage by freezing;‖ (*b*) light weights to handle; (*c*) ease of laying in wet trench; (*d*) facility of repairs.¶ Fire tests on exposed pipe indi-

* See series of articles by Andrew Swickhard, in *E. C.*, Nov. 4, Dec. 2, 1914, and Jan. 6, Feb. 17, and June 2, 1915.

† Can be laid with less cover than metal pipe since its resistance to freezing is greater.

‡ Some makers will erect a pipe under a guaranteed leakage limit of 100 gals. per day per in. mi.

§ See also *E. N. R.*, Aug. 12, 1920, p. 309, and *J. A. W. W. A.*, Vol. 11, 1924, p. 564.

‖ Damage by freezing at Everett, Wash., see *E. N. R.*, Dec. 31, 1925, p. 1077.

¶ See *E. N.*, Apr. 6, 1916, p. 647.

cate high resistance to destruction.[20] Ledoux[5a] compared first costs per lin. ft. for pipes under 65 lb. pressure, New York delivery, 1922, (cast-iron pipe, at $44 per ton; trenching taken constant for one diam.; paving costs ignored; hauling at $2 per ton) with results shown in Table 93. For calculations involving pumping charge, see *E. N. R.*, Nov. 11, 1920, p. 932.

Table 93*

Diam., in.	Machine-banded wood pipe		Continuous wood pipe, redwood	Cast-iron pipe
	Pine	Redwood		
6	$0.96	....	....	$1.26
12	1.49	....	....	2.09
20	2.29	$3.70	$3.40	5.21
24	2.89	4.84	4.19	6.71
30	4.13	....	5.27	9.13
36	....	....	6.35	12.02
48	....	....	9.50	18.90

* See also Maury's comparisons, *J. A. W. W. A.*, Vol. 12, 1924, p. 1.

Disadvantages, besides excessive leakage and short life, are: (*a*) On large pipes, over 3-ft. diam., a great depth of backfill with low pressures will cause shortening of vertical diam., decreasing capacity. (*b*) Excessive leakage under varying pumping pressures; pipe at Porterville, Cal., had to be replaced in 3 months.[41] Leakage results from deformation of staves when pipe is emptied and refilled. (*c*) Hydraulic Power Comm., N. E. L. A., would restrict use, except for construction works, to heads below 100 ft.* (*d*) Danger of collapse under heavy fills; the engineering report on a 36-in. pipe collapsing under a 10-ft. fill cited 1½-in. staves as good for 2-ft. fill.[6] A 60-in. pipe collapsed at Deer Creek, S. D., in 1915 under 2-ft. fill.[7]† At Seattle a 42-in. pipe collapsed under poorly placed backfill which allowed boulders to press unevenly on pipe.[8] (*e*) Air valves are a necessity (see p. 446; for type of vault see *E. N. R.*, June 14, 1917, p. 568). (*f*) On some installations, there is not a sufficient factor of safety. Staves failed at Vancouver when the 36-in. conduit, 16 years old, was subjected to 25-ft. additional head.[40] (*g*) At various cantonments and other war projects, minor troubles were encountered with factory-made pipe. At Old Hickory plant[9] occasional failures developed from defective bands; warping of staves and shrinkage caused by exposure to weather for several months were quite serious. In some cantonment work, the tenons on 8-in. pipe would not fit the cast-iron specials on hand; this was remedied by forming a new spigot.[10] Some of these troubles were undoubtedly due to rush work in war time.

Wood vs. Steel Pipe. In wood pipe the metal is for strength only; while in a steel pipe, the metal serves both for strength, and for a water stop. A steel pipe might leak excessively when only 5 per cent. of its section has been destroyed, while a wood pipe might continue tight and the bands would not be stressed beyond limit until 75 per cent. of their cross-section had rusted away. This is a great advantage in soils hostile to steel; round bands are

* Butte Water Co. uses wood-stave pipe for heads below 300 ft., and lap-welded steel for high heads.[43]

† At Memphis, Continental Pipe Mfg. Co. built a 60-in. pipe under a 20-ft. fill.

better able to resist corrosion than thin sheets, from which steel pipe is made. Steel pipe is subject to corrosion and pitting on both inside and outside. On wood pipe, circular bands are of greater diam. than the thickness of steel pipe for same service, and, in addition, may have added protection of galvanizing and heavy dip of tar or asphaltum. No continuous stave or machine-banded wood pipe has yet been reported attacked by electrolysis. Metal pipes have been replaced by wood where conditions were severe. A 36-in. steel penstock at Bangor, Penna., was replaced by continuous pipe of white pine.[11] Steel pipes have far less leakage, particularly under pumping pressures.

CONTINUOUS-STAVE PIPE*

Construction. Continuous-stave wood pipe is adapted to diam. of 12 to 192 in.[42]† or even larger. Staves of selected lumber are milled to assemble in the field with the interior diam. desired, staves of various lengths being laid side by side so that end joints are staggered. Connections at ends of staves are made by inserting metal tongues‡ in saw kerfs, or slits, accurately cut, in exactly the same position in each stave, so that ends of staves will be even on both inside and outside of pipe—whence the designation "continuous stave." In the most modern form substantial tongues and grooves are milled on sides of staves (see Fig. 160) to assist in retaining the circular form as well as to obviate the necessity of hammering the wood to round out the pipe; bruised wood has a tendency to decay. The staves are clinched firmly together by bands of round steel rods with upset ends and cold-rolled threads, the ends of each band being held together by malleable iron shoes stronger than the rod.

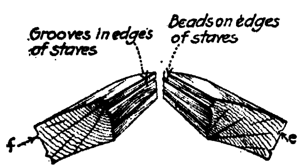


FIG. 160.—Isometric of two staves. e = grooves in edges, $\frac{1}{8}$ in. deep, $\frac{1}{8}$ in. wide; f = beads, $\frac{3}{16}$ in. high, $\frac{3}{16}$ in. wide at base, $\frac{3}{8}$ in. from outer and inner face. When staves are banded together, the bead, being a little larger than the groove, is squeezed into the latter.

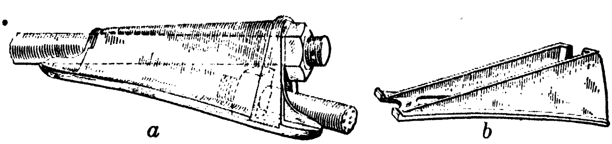
Woods. The only woods regarded as generally suitable are redwood, fir,§ white pine, Norway pine, and cypress.[5b] High pressures require close-grained wood of slow growth, free from knots and other defects; high tensile and compressive strengths are essential. Commercial-run lumber should not be used, except in temporary pipes. Redwood and fir‖ grow to great sizes and it is easier to obtain from them than from eastern pines and other woods, clear material.[5b] Pitch pine (*Pinus rigida*) was used for bored logs in Philadelphia; a piece dug up at Washington Square, after a century, was in "excellent condition."[23] Local spruce, air dried for 9 months, was used on upper end of 60-in. line at Pittsford, Vt.[24] White pine staves were used for a penstock at Bangor, Maine, which were selected log run from West Virginia, grading No. 2 barn and better.[25] Local hemlock was used in North Carolina[26] for 30-in. line 1200 ft. long under 144-ft. head. Material for some large eastern pen-

* For design, see p. 371.
† For 192-in. (16-ft.) conduit of California and Oregon Power Co., see *E. N. R.*, Jan. 14, 1926, p. 68.
‡ Malleable cast-iron butt joints were used by Kelsey at Logan, Utah and Deadwood, S. D.
§ Douglas fir (*Pseudotsuga taxifolia*) is known also as Washington fir and Oregon pine.
‖ Fir can be subjected to a bending stress of 800 lbs. per sq. in., redwood to 600 lbs.

stocks was of Douglas fir staves, 3½ in. thick, sawed, planed, and milled in Pacific Coast plants.[27]

Cypress makes durable pipe, and is the only competitor of redwood in length of life and endurance under alternately wet and dry conditions.* If cypress is selected to eliminate sap, it probably is as long-lived as redwood. Disadvantages are: (*a*) Quantity of standing timber is extremely limited and cost is high.[21b] (*b*) It is difficult to get cypress for pipes, because of the sap; where sap is eliminated, price is so high that it cannot compete. Cypress pipes are rare.[21b]

Laying. It is very important to have the butt joints driven tight, as practically all leakage is at these joints. Staves must be trimmed exactly square on the ends for tightly driven butt joints.[28] Vertical and horizontal curvature can be made without difficulty, radius depending on diam. of pipe—in general, 60 diams. of pipe (except in largest sizes, which require a greater radius), but sharper curves have been built. A 30-in. line for Santa Barbara, Cal., had curves on a radius of 120 ft. Contractors have found that continuous pipe laying requires a skilled organization and that there is economy in subletting such work to the pipe companies. For methods and equipment, see *E. C.*, June 2, 1915, p. 483.



a *b*

FIG. 161.—*a*, Malleable iron shoe for round bands with straight pull on pipe lines, tanks, etc., Racine Tank Lug Co., Racine, Wis.; *b*, malleable shoe for wood pipe, Marion Malleable Iron Works, Marion, Ind. Shoes of the Allen type are specified by Partridge.[21a]

Trench should be 2 ft. wider than inside diameter of pipe, should be backfilled with clean earth free from vegetable mold, roots, grass, sod, or other humus material, and should be thoroughly tamped, up to approximately center line of pipe. Loose rocks should never be put into trench until pipe has been well covered with earth.

Shoes are an important feature, and only tested types should be used. Shoes must be capable of developing the full strength of the band. Disregard of this resulted in the waste of 300 tons on one line.[21c] Double shoes are used on pipes above 48 in., single shoes on smaller sizes.

Foundations.[30] When pipe is above ground, cradles are used to secure free circulation of air; in a dry climate this assures longer life. Foundations for 60-in. power intake consisted of concrete cradles 8 ft. 9 in. apart, 6 in. thick above ground, with footings 12 in. thick extending to rock or solid earth. Foundations require considerable concrete or heavy timber construction. The latter requires creosoting or occasional replacement (see p. 369). See construction methods in *E. N. R.*, May 30, 1918, p. 1060.

Lynchburg, Va. conduit, built 1906, consists of wood-stave pipe for pressures up to 83 lb., steel pipe for higher pressures, and cast-iron pipe for stream crossings, all 30 in. in diam. Profile is such as to keep wood pipe always under

* Some engineers recommend only creosoted pipe for such condition.

Table 94. Details of Design for Continuous-stave Pipe, Classes A, B, and C314

Size of pipe, in.	Stave thickness, in. Standard	Stave thickness, in. Maximum	Number of staves to circle Standard	Number of staves to circle Maximum	Stock size, lumber, in. Standard	Stock size, lumber, in. Maximum	Number ft. b. m., per ft. of pipe* Standard	Number ft. b. m., per ft. of pipe* Maximum	Top width of stave, in. Standard	Top width of stave, in. Maximum	Size of band, in in.	Number of pieces in band	Spacing of bands,† in. For 100-ft. head	Radius of curvature to which pipe can be built, in ft.
12	1⅝	1⅝	13	13	2×4	2×4	8⅔	8⅔	3 560	3.679		1	6 38	60
14	1⅝	1 11/16	15	15	2×4	2×4	10	10	3 512	3.642	⅜	1	5.45	70
16	1 7/16	1 11/16	17	17	2×4	2×4	11⅓	11⅓	3.496	3 588		1	4.76	80
18	1 7/16	1 11/16	18	19	2×4	2×4	12	12⅔	3 659	3 545		1	5.76	90
20	1 7/16	1 11/16	20	20	2×4	2×4	13⅓	13⅓	3.617	3.688		1	5.20	100
22	1 7/16	1 11/16	22	22	2×4	2×4	14⅔	14⅔	3.572	3.643		1	4 73	110
24	1 7/16	1 11/16	23	24	2×4	2×4	15⅓	16	3.695	3.604	7/16	1	4 34	120
26	1½	1 9/16	17	17	2×6	2×6	17	17	5.374	5.397		1	4.00	130
28	1½	1 9/16	18	18	2×6	2×6	18	18	5.432	5.453		1	3 72	140
30	1½	1 9/16	19	19	2×6	2×6	19	19	5.477	5.498		1	4 53	150
32	1½	1 9/16	20	20	2×6	2×6	20	20	5.525	5.545		1	4.25	160
34	1½	1 9/16	21	21	2×6	2×6	21	21	5.562	5.580		1	3.98	170
36	1 9/16	1⅝	22	22	2×6	2×6	22	22	5.620	5.638		1	3.77	180
38	1 9/16	1⅝	23	23	2×6	2×6	23	23	5.657	5.674	½	1	3 57	190
40	1 9/16	1⅝	24	25	2×6	2×6	24	25	5.682	5.471		1	3 39	200
42	1⅝	1 11/16	26	26	2×6	2×6	26	26	5.513	5.516		1	3 23	210
44	1⅝	1 11/16	27	27	2×6	2×6	27	27	5.536	5.551		1	3.09	220
46	1⅝	1 11/16	28	28	2×6	2×6	28	28	5.570	5.580		1	2.96	230
48	1⅝	1 11/16	29	29	2×6	2×6	29	29	5.596	5.615	½, or ⅝	1	{2.84 4.41}	240
50	1⅝	1 11/16	30	30	2×6	2×6	30	30	5.618	5.632		1	4 24	250
52	1⅝	1 11/16	31	31	2×6	2×6	31	31	5.652	5.662		1	4 07	260
54	2½	2 11/16	33	34	3×6	3×6	49½	51	5.658	5.528		2	3.92	270
56	2½	2 11/16	34	35	3×6	3×6	51	52½	5.678	5.552		2	3 80	280
58	2½	2 11/16	36	36	3×6	3×6	54	54	5.542	5.574		2	3.66	290
60	2½	2 11/16	37	37	3×6	3×6	55½	55½	5.563	5.595	⅝	2	3.54	300
62	2½	2 11/16	38	38	3×6	3×6	57	57	5.584	5 615		2	3.42	310
64	2½	2 11/16	39	39	3×6	3×6	58½	58½	5.604	5.634		2	3 32	320
66	2½	2 11/16	40	40	3×6	3×6	60	60	5.622	5.562		2	3.22	330
68	2½	2 11/16	41	41	3×6	3×6	61½	61½	5.640	5.669		2	3.12	340
70	2½	2 11/16	42	42	3×6	3×6	63	63	5.657	5.685		2	3.03	350
72	3½	3 11/16	44	45	4×6	4×6	88	90	5 685	5.585	⅝, ¾	2	{2.95 4 24}	430
74	3½	3 11/16	46	46	4×6	4×6	92	92	5.579	5 604		2	4 12	440
76	3½	3 11/16	47	47	4×6	4×6	94	94	5.594	5.620		2	4 02	450
78	3½	3 11/16	48	48	4×6	4×6	96	96	5.610	5.634		2	3 92	470
80	3½	3 11/16	49	49	4×6	4×6	98	98	5.625	5 649		2	3 81	480
82	3½	3 11/16	50	50	4×6	4×6	100	100	5.640	5.663		2	3.72	490
84	3½	3 11/16	51	51	4×6	4×6	102	102	5.654	5.677		2	3.63	500
86	3½	3 11/16	52	53	4×6	4×6	104	106	5.667	5.583		2	3.55	516
88	3½	3 11/16	53	54	4×6	4×6	106	108	5.681	5.596	¾	2	3.48	528
90	3½	3 11/16	55	55	4×6	4×6	110	110	5 588	5.009		2	3 39	540
92	3½	3 11/16	56	56	4×6	4×6	112	112	5.602	5.623		2	3 32	552
94	3½	3 11/16	57	57	4×6	4×6	114	114	5.616	5 637		2	3.25	564
96	3⅝	3 11/16	58	58	4×6	4×6	116	116	5.642	5.649		2	3 18	575
108	3⅝	3 11/16	65	65	4×6	4×6	130	130	5.620	5.626		2	2 83	750
120	3⅝	3 11/16	71	72	4×6	4×6	142	144	5.681	5.614		2	2 54	1080
132	3⅝	3 11/16	78	78	4×6	4×6	156	156	5.659	5.669		2	2 32	1190
144	3⅝	3¾	85	85	4×6	4×6	170	170	5.642	5.651	¾, ⅞	2	{2.12 2.89}	1300

* Allowing for waste.

† Ma imum allowable spacing of bands is 10 in. for all, except 8 in. on 96 to 144 in., and on 72 in. with ⅝-in. band; and 6 in. on 144 in. with ⅞-in. band.

pressure. Trench is 6 ft. deep. Minimum radius = 200 ft. California redwood staves, 2 by 6 in., 10 to 24 ft. long, surfaces planed to circumference. One radial face of stave has ⅛-in. bead, 1/16 in. high, which under pressure is forced into the plane radial face of the adjacent stave. Leakage at ends of staves is prevented by ⅛-in. steel plates fitted into saw kerfs. Bands are

½-in. mild steel rounds, specified to have ultimate strength of 58,000 to 65,000 lb. per sq. in. Each band has hemispherical head on one end and cold-rolled thread on other so proportioned that rupture will occur in rod; all tests have proved the adequacy of the proportion. Bands are spaced by formula $N = 3.3H$, applicable only to 30-in. pipe under conditions of the specifications, where N is number of bands per 100 ft., H is pressure head in feet. Bands engage malleable cast-iron shoes, so designed that friction on the base will allow slipping around pipe, rather than cutting into wood, during tightening. At least ⅛ in. of metal in all parts of shoe. While stored, awaiting use, weather caused cracking and checking of ends of staves. Contractor sawed off damaged ends by hand. It was found impossible to do hand sawing accurately enough to make joints as tight as desired. Bands and shoes were dipped in hot Pioneer mineral-rubber coating, and touched up with Smith's "durable metal coating." Bends were made by springing pipe while loosely banded, driving butt joints tight, when bands were cinched. Heavy rains before backfilling caused pipe to float 5 or 6 ft. in some instances.[31] Parts of it stood empty 3 years after construction; in 1924, the city manager reported that 27 leaks were repaired in past year, involving replacement of 154 staves. The flow was interrupted four times. While draining a section, 100 ft. of pipe at a high point, ½ mi. distant, collapsed due to decay of staves to such an extent that they could not resist the unbalanced atmospheric pressure.

MACHINE-MADE* WOOD PIPE (SECTION PIPE)

Kinds. Made in lengths from 3 to 20 ft., and of diam. from 2 to 24 in., occasionally as large as 48 in., from same quality of selected lumber as continuous stave (see p. 360), though generally of a less thickness. Wood is purchased in mill sizes 2 × 4, or 2 × 3 in., of Nos. 1 or 2 clear; generally kiln dried for the smaller sizes. Air seasoning, where practicable, is more satisfactory. Pipes made on Pacific Coast are generally *wire wound** under tension, with a continuous heavily galvanized wire having a tensile strength of 60,000 to 65,000 lb. per sq. in. Pipes made elsewhere in the United States are generally *machine banded*, a continuous band of steel of about No. 16 gage being wound under tension. Section pipe is also made with individual rods and lugs. The stave thickness is the same for all pressures, but the size and spacing of metal members are varied to conform to heads from 50 to 400 ft., by 50-ft. increments.

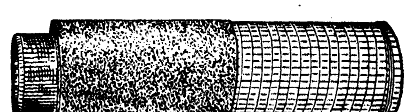


FIG. 162.—Machine-made wood-stave pipe for low pressure. (Protective coating omitted at right to show steel wires. Winding is doubled at end.)

Manufacturing. The wires or bands, while free from rust, are run through a bath of warm asphaltum and then wound spirally upon the pipe, carrying a heavy coating, thus preventing the inner surface of the band from coming in direct contact with the outer surfaces of the staves. As soon as banding is completed, the pipe is run over two rolls moving through a warm bath of

* Also called "machine banded," in loose terminology.

asphaltum, thus coating the outside $\frac{1}{4}$ to $\frac{3}{8}$ in. thick. To prevent coating from running, the pipe is immediately rolled in sawdust, which adheres and prevents it being abraded in handling. Extra coat can be applied by repeating the operation. Coating, as soon as cool, becomes tough, and can be removed only by a chisel (for coating, see p. 371).

Joints. Sections are joined in various ways, each of which has merits. Cheapest, well adapted to low gravity heads, is the inserted joint (also known as "mortise-and-tenon" and "chamber-and-tenon," Fig. 163), formed by cutting a chamber in one end to half the thickness of the stave, and forming a tenon in the other. This joint insures a thorough saturation under low pressures and is used by many eastern factories (El Paso & Southwestern Ry. Pipe, 110 miles long,[13] is mostly of this type). Wood coupling is advocated by many western makers; for low pressure it is wire wound; for high pressures, reinforced with rings and shoes; the latter is recommended above 16 in. No cutting of ends of pipes into mortise and tenon is required, but

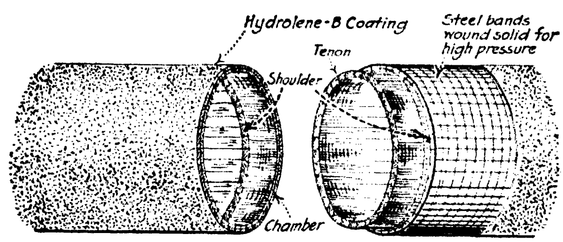


FIG. 163.—Ends of machine-made wood-stave pipe.*

many lines have had trouble due to decay of collars and pipe ends which do not become saturated. Durability is promoted by use of creosoted collars. Some types made by Am. Wood Pipe Co. for mine drainage have wires set in grooves to lessen chance of injury. Cast-iron couplings are used for high pressures; although costly, they are considered a good investment by many engineers, among them J. L. Campbell.[13]

Spacing of Wires. Closely spaced small wire has been found more satisfactory than widely-spaced coarse wire, both in holding the staves to shape, and in preserving the galvanizing. The Western Union splice used requires a small loop, the forming of which cracks off the coating of a coarse wire. The wire is fastened at the end by clips.

Spacing of Bands.[23] From experiments, it was found that a factor of safety of 3 was ample for the steel bands used. Pipes subjected to pressure of 80 lb. or over and 12 in. in diam. or over, for municipal waterworks, are solidly wound; that is, they are steel pipes with a wooden lining. The band is secured to each end of the pipe by double winding after the end is fastened with two or more screws, and on all high-pressure pipes screws are put through the band into the wood every foot or 8 in. apart, according to diam. and pressure.

Eastern vs. Western Practice. Eastern manufacturers, as typified by Michigan Pipe Co., Standard Wood Pipe Co., and A. Wyckoff & Son Co., use white pine, tamarack, cypress, Douglas fir, and redwood; the pipe is

* Another type involves a wooden collar, or coupling.

generally machined-banded pipe, with inserted joints. Western manufacturers, *e.g.*, American Wood Pipe Co., Continental Pipe Mfg. Co., Pacific Tank & Pipe Co., and Redwood Manufacturers Co., employ either Douglas fir or redwood, and make wire-wound pipe with couplings at the joints. For small systems, experience in New Hampshire[34] indicates eastern product to be superior for local conditions both as to durability and resistance to water hammer.

Flat Bands vs. Wires. When pipe is subjected to a pressure fluctuating over a 50-lb. range, the higher pressures force the wires into the wood, and when the lower pressures obtain the joints open, permitting leakage. Bands present a larger surface to the wood, and have less tendency to sink in and allow leaks. Bands can also be coated more effectively. On the other hand,

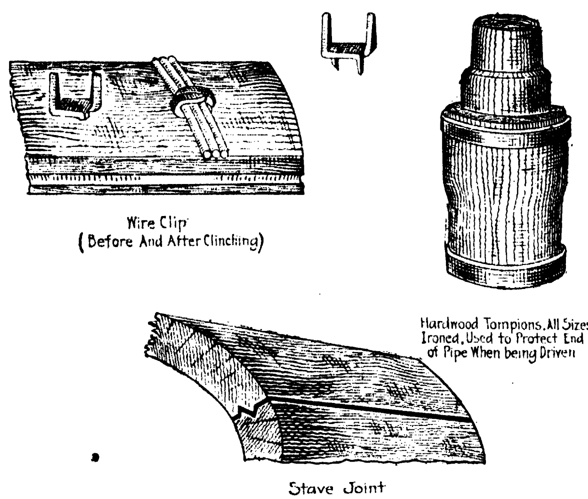


FIG. 164.
(Pacific Tank and Pipe Co.)

wire presents the minimum surface to corrosion, while a band presents the maximum. The manufacture of wire assures a better quality of metal than in bands.

Continuous vs. Machine-made Pipe.—Investigation of continuous-stave pipe led T. Chalkley Hatton[2a] to the belief that there were too many uncertainties in manufacture; construction must be left to irresponsible, partly unskilled workmen, who in many instances had to work under unfavorable conditions. For these reasons machine-made wood-stave pipe is superior, being made and banded by automatic machines run by capable workmen, staves, bands, coating, and workmanship being open to inspection under most favorable conditions. On the other hand, continuous-stave pipe has cheaper freight rates and lends itself more easily to rigid inspection; also wood less thoroughly seasoned can be used.

Pipe Laying. Pipe should be carefully laid, no matter what the pressure is to be. Examine each end as the sections are driven together, and turn the joints to bring any bruised or scratched places to the upper side, as there may

be small leaks at these places, which can be more easily plugged if on top. By the use of tompions furnished by the manufacturers, and an improvised ram about 5 ft. long, ironed at one end, laying is easy. Pipe 6 in. and larger should be driven together with a ram; smaller sizes can be driven with a maul. Keep the alinement as true as possible, and drive until the shoulders come together at every point. The pipe is usually driven from the coupling end of wood-coupling pipe, or the female end of inserted-joint pipe. If necessary to cut the pipe, mark the place and drive at least two strands of wire together far enough back on each side to allow the required length of tenon. Staple the wire securely before cutting. If necessary to install fittings in an inserted joint line, cut a length of pipe in the manner described at some point in the body. This will save the trouble of making a new tenon on the end, and the ends made by cutting can be made to fit perfectly into the bell of the fitting by rasping. Tenons on wood-sleeved pipe fit the bells of the fittings, but this pipe is best handled as above, when the fitting comes midway in a length. In taking outlets from a line already laid, a saddle can be used; attach it to the pipe by cutting the wire after stapling, and stripping for a space the width of a saddle. Then fit the saddle to the pipe, and mark the point at which the opening is to be cut. After the opening is cut, place a rubber gasket under the saddle and attach with U-bolts or bands.*

Curvature. It is better, if conditions permit, to make a required deflection by the use of a long smooth curve by throwing a slight angle in each joint than by a special fitting. Wire-wound pipe will permit of 2 to 6° deflection in each joint, depending on the size, service head, and skill with which laid. Smaller sizes can be deflected more than the larger, and low-pressure pipe more than high. When vertical or horizontal curves are to be built, include a number of short lengths in the order. After laying a curve, tamp in enough backfilling on the outside of the curve to keep it from buckling and then ram the joints together tight by using the tompion and ram on the last length. Extra care should be taken in backfilling around a curve to see that the tamping is thoroughly done, as there is always a tendency, when the pipe is under pressure, for it to "work" and blow out on curves. Fittings, particularly bends, should be thoroughly anchored, by placing a strut against the side of the trench, driving in a strong stake and wedging tight against the fitting, or by means of a large stone. Take care that the anchor is not misplaced in backfilling. Plugs should always be held thus, for, while they might hold themselves after becoming soaked, they are apt to blow out when dry.

Design. Formulas of p. 371 apply. See diagrams in *E. N. R.*, Oct. 9, 1919, p. 710, and in "Working Data for Irrigation Engineers," by Moritz, (Wiley, 1915).

Repairs to a 24-in. machine-banded pipe at Norfolk, which leaked badly under normal pressure after operating at subnormal for several years, consisted in placing a rubber band, 1½ × ¼ in., over each joint and clamping it to tightness by a ⅝-in. rod (with shoe), placed outside. Rubber band was omitted under the shoe; here the opening in the mortise-and-tenon joint was calked.[35] Leakage was reduced from 2500 to 360 gal. per in. mi. per day.[12c]

* See methods employed on 9 mi. of 8-in. pipe, *E. C.*, Feb. 18, 1914, p. 221.

Machine-made Pipe for Distribution Systems. From experience on cantonment work, some engineers state the chief difficulty lies in making connections either to valves or for service pipes; one engineer concludes that "it is not well suited to distribution work where specials and valves must be used liberally."[34] Where few house connections are required, present or future, and when there is a constant pressure which does not exceed 200 lb., wood pipe offers advantages over cast-iron. It has been used successfully on several New England systems.[34] At Antrim, N. H., built in 1896, head exceeds 200 ft. There is "infinitesimal leakage;" in perfect condition after 20 years. At Campton Village, N. H., 12-, 10-, 8-, and 6-in. pipe installed in 1905; head, 186 ft.; no leaks; no expense for repairs. At Freedom, N. H., head 300 ft., installed, 1912. All pipe is in trenches.[34]

Camp Dix, N. J.[36] Redwood, shipped from Pacific Coast (60,000 ft.); size, 6 to 16 in. *Trench:* minimum cover, 3 ft.; width, only slightly larger than diam. of pipe, at places not more than 4 in. Machines used, Parsons, Austin, Buckeye. Two men drive spigot end into socket, swinging a heavy log, used as a maul, from a rope, as the men stand on sides of trench. Connections for hydrants were ordinary cast-iron specials, connecting to special specials (bell-and-spigot pieces about 2 ft. long). These bells have straight, smooth inner surfaces. Service connections are made by inserting into the wood an ordinary corporation cock made with a wood screw. Slight line curvatures are made by wood pipe bends.

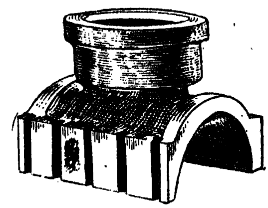


FIG. 165.—Saddle or split tee.

Cast-iron Fittings. Cast-iron collars can be furnished tapering out to the end from the center of the inside, into which the ends of the pipe can be driven. Machine-banded pipe can be fitted to standard cast-iron fittings having bell or hub ends, but makers furnish lighter fittings, amply strong, and smooth finished in the bell, at less cost, because lighter, and which save labor in fitting the pipe.

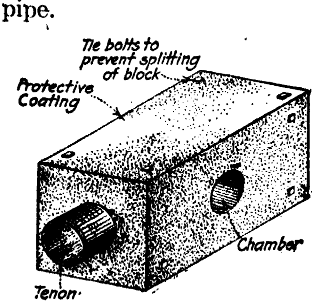


FIG. 166.—Wood elbow for wood pipe, 6 in. and smaller.

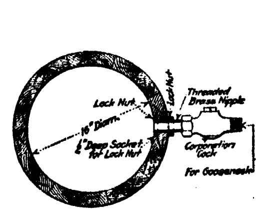


FIG. 167.—Service connection.[37]

Service Connections to Wood Pipe. Bore a hole in middle of a stave with a common wood bit $\frac{1}{32}$ in. smaller than outside diam. of thread on nipple or corporation cock. Connection is then screwed in until its inner end is flush with inside of pipe. When making taps under pressure, bore only until screw on bit goes through and water begins to show; connection can then be screwed

in far enough to splinter off the thin piece of wood. In this manner taps can be made without drenching the workmen, but a service connection should have cock or valve attached before it is inserted, to hold water while rest of service is being coupled. Blow out thoroughly before connecting to house service to make sure connection is screwed in far enough to get full opening. An extension bit is convenient, as size can be adjusted to a nicety.

Comparative Cost. The cost of laying machine-made pipe is less than for any other pipe for water under pressure; there is no special labor or extra material used in making inserted joints; the width of excavation is less because no joints are to be made and it is not requisite that men should get around the pipe in laying it; the pipe is light enough to permit being laid without the slow-moving block and fall. Even a 48-in. pipe can be lowered and fitted by eight men with four rope slings, and six of these men can drive the pipe home. In wet trenches the cost of pumping is reduced.

Table 95. Dimensions, Transportation, Hauling, and Laying of Wood Pipe[33]

Size of pipe in inches	Outside diameter of pipe in inches	Weight per lineal foot in lb., 80 lb. pres.	No. feet in carload (40-ft. car)	No. feet 2 horse truck can haul	No. men used in laying, exclusive of foreman	No. feet these men can lay in day of 10 hr.
6	$10\frac{1}{8}$	13	3100	344	4	2000
8	$12\frac{1}{8}$	16	2600	272	4	1900
10	$14\frac{1}{8}$	19	2100	216	4	1700
12	$16\frac{1}{8}$	23	1600	192	4	1500
14	$18\frac{1}{8}$	26	1200	152	6	1400
16	$20\frac{1}{8}$	30	1000	144	6	1200
18	$22\frac{1}{8}$	33	800	128	6	1000
20	$24\frac{1}{8}$	36	700	112	6	800
24	$28\frac{1}{8}$	45	500	80	8	600
30	$36\frac{1}{8}$	70	275	64	8	450
36	$42\frac{1}{8}$	100	160	48	8	300
48	$54\frac{1}{8}$	140	100	24	8	200

DESIGN AND OPERATION

Leakage depends on care in construction and the maintenance of constant pressure. Wood pipe is naturally more porous than metal. Hazen tested a factory-made section to 200-lb. pressure, which sweated all over, but there were no jet leaks;[12a] the manufacturer considered it tight. Leakage between staves is increased when pressure forces the rings into the wood; this allows the bore of the pipe to become larger. Hazen could not get a line tight under 150-lb. pressure because the required hoop tension crushed the fibers.[12a] Pumping pressure variations, and emptying and filling, result in leakage. Leaks in 30 mi. of distribution mains at Spokane, Wash., according to Hodgman,[12b] occurred "too fast to stop," under pressures fluctuating from 18 to 90 lb. (see also p. 359). El Paso line, 110 mi. long, was tested when 12 years old. A machine-made section, 11.5 mi. long, with mostly inserted joints and aver. equated diam. of 17.29 in., leaked 15 gal. per in. mi. per day. A second section, 28.2 mi. long of similar type, but with an average diam. of 9.64 in., leaked 262 gals. per in. mi. per day.[13] Leakage allowed in specifications depends on conditions. Norfolk machine-made 30-in. line under static pressure tests indicated a leakage of 310 gal. per in. mi. per day, as against 83

gal. for lock-joint concrete pipe.[4] Ledoux cites as a low leakage 500 gal. per in. mi. per day.[14] A 30-in. line at Suffolk, Va., was accepted by Quartermasters' Dept., U. S. A., in 1919 when leakage had been reduced from 1070 to 500 gal. per in. mi. per day.[15]

When placing new pipe in service, allow several days for pipe to soak and swell. On cantonment work leakage was reduced by filling the new line at a pressure or 1 or 2 lb., and allowing it to soak 2 or 3 days before increasing to working pressure.

Durability is dependent on metal bands, susceptible to corrosion (see p. 809), and wooden staves, subject to decay. Decay is a communicable fungus converting the wood substance and cell contents into food on which the fungus thrives. Air, water, and heat are essential to growth.* Extreme cold only retards decay, but heat above 150°F. stops it. A buried pipe will be protected from air and heat to some extent, and will be exposed to more constant moisture. Conditions that shorten life are intermittent filling; heads below 15 ft.; gravelly or open soils; alkali soils; cinder fills; contact with mine waters; and, on machine-made pipe, collars which fail to become saturated.[16] Conclusions by *D. C. Henny*,† from 30 years' experience, are:

Redwood staves under constant pressure show little depreciation when buried (pipe line laid for Butte W. W., in 1892, in perfect condition in 1921), and show slow decay when exposed for a large portion of the time to direct sunlight. When running partly empty decay is more rapid, either when buried or exposed. Fir staves are liable to decay, slowly when exposed or when buried in tight soil free from vegetable matter, but rapidly when buried in open soil. Paint or a tar coating will retard decay. Creosoted fir staves have been used since about 1912. Experience is too short to justify conclusions, but probably creosoting puts fir staves on a par with untreated redwood. These statements apply to sound wood only; the presence of sapwood hastens decay. It is becoming common practice, especially for large power pipe lines, to leave them exposed, supported on concrete saddles which makes inspection and repair easy. Round steel rings and malleable iron pipe shoes are usually well coated with asphaltum, and suffer little corrosion, even when buried.‡ No discontinuance is on record caused by excessive weakening by corrosion. The same is true of steel tongues inserted in butt joints.

Table 96. Approximate Cost of Laying Pipe*
Excluding Hauling, Excavation, and the Pipe Itself

Size pipe, inches	4	5	6	8	10	12	14	16	18	20	24	30	36	48
Wooden	$0.02	$0.02	$0.02	$0.04	$0.06	$0.06	$0.08	$0.08	$0.08	$0.10	$0.12	$0.14	$0.16	$0.20
Steel-riveted	....	....	....	....	0.60	0.60	0.64	0.68	0.72	0.82	0.96	1.20	1.64	1.82
Cast-iron (including lead and hemp)	0.18	....	0.28	0.36	0.42	0.50	0.54	0.72	0.78	1.02	1.28	1.60	2.18	2.60

Lead and Hemp at 10 cts. per lb. Labor at 50 cents per hour.
* Catalog of A. Wyckoff & Son, Co., Elmira, N. Y. Similar claims in many catalogs of manufacturers of wood-pipe. See also p. 405.

* For life of timber set under conditions favorable to decay, see 1909 *Report*, National Conservation Commission.
† See also *E. N.*, Aug. 26, 1915, p. 400.
‡ Some pipes have been encased in concrete.

Arthur L. Adams[2b] cautions against attributing lack of durability to use of fir for staves instead of redwood, as an important redwood pipe line in Southern California, built about same time, had not shown much, if any, better results.

T. Chalkley Hatton[2a] investigated (1907) wood pipe of irrigation work in West where continuous stave pipe has been used many years, and several municipal and industrial plants where both continuous and machine-made pipe has been in use at least 40 years. Continuous stave pipes where bruised either in handling or by cinching bands too tightly, in most instances suffered decay where pressure was less than 50 lb., also where longitudinal joints were wide at outer surface and contact not good, or at end joints not in close contact, or where a dry crack existed in end of stave before it was built into pipe. In many places where pipes were examined material in contact was at least 50 per cent. vegetable matter, and no evidence of decay was found, although pipes had been in constant use 8 to 14 years; where top of pipe was only partly covered, or had but few inches of dry earth over it, dry rot had extended to depth of ⅛ to ¼ in., but as soon as constant saturation was reached wood was sound. Greatest defects were in end joints, due to saw kerfs not being exactly in position, so that when metal tongue was inserted inner or outer edge of one stave was a little higher or lower than that of its neighbor. Another marked defect was in durability of steel bands. These bands are ¼ to ¾ in. in diam., coated with asphaltum pitch or similar material, and supposed to be free from oxide when placed. Hundreds of bands when delivered had been exposed to weather sufficient time to give them a thick coat of rust. In this condition they were dipped, covering rust but not stopping its deteriorating effects. In many instances threads were badly rusted, not having been coated; and in most instances it would have been dangerous to move the nut, as stripping would have occurred.

The Continental Pipe Co.[28] concludes from 32 years' experience that (1) there is little difference in life of well-constructed wood pipe of proper material above or below ground, any difference showing only after many years; (2) wood pipe will last longer under comparatively high pressure than under low head, owing to greater saturation (there are many wood-pipe lines under practically no head in good condition after 20 to 25 years' service); (3) life of buried pipe will depend somewhat on soil conditions.

Life. Data from questionnaire (1921) of U. S. Reclamation Service[16] on 196 wood pipes, many of which were full part time only, and some in alkali soils: Three pipes, of untreated fir staves but 4 years old, reported in poor condition; in wet soil, under but 15-ft. head, and full but part time. Of pipes installed 1911–1915, 85 per cent. reported fair or better; bands on one redwood pipe destroyed by alkali soil. Of pipes installed before 1910, 76 per cent. in fair or better condition. From these data it was concluded that with some maintenance the average life should be 15 years. The Comm. on Depreciation of A. W. W. A. reported:[17] "Ultimate experience somewhat limited, but thought to be in same class with steel; when well protected, and constantly saturated, 30 to 60 years." In a valuation for Eureka, Cal., State Railway Comm., placed life of a redwood distributing system at 35 years.[18] Ledoux[14] places life of a continuous pipe, properly designed and laid, and kept filled with water, at 25 years, while good uncoated material alter-

nately dry and wet will last but 12 years.[5a] A pitch-pine pipe was dug up in Boston after 130 years (used 1789–1840) and found rotted on the outside only.[19] A 36-in. line built in cinder fill along the Pennsylvania R. R. gave trouble after 2 years, due to corrosion of the metal.[12d]

Creosoted staves and machine-made pipes are now obtainable. For methods of creosoting, see *E. N. R.*, Oct. 4, 1917, p. 639. These offer: (1) longer life;* (2) less loss of water when first filling; (3) less tendency for bands to cut into the wood; (4) less maintenance; (5) cinching tight can be done when erecting continuous lines, as no allowance is required for swelling. Manufacturers claim that creosoted pipe does not affect taste† of water.[20] Horricks recommends straight distillate creosote, to avoid contamination of the water.[46] Creosoting adds approximately 15 to 20 per cent. to the cost.

Coating can afford protection to a degree only. Henny[16] concludes that coating should be continuous and heavy, not less than $\frac{1}{16}$ in. to be fully effective, and should preferably consist of more than one coat of a mixture of asphaltum and tar, or of an application of gas tar followed by refined coal tar. Little experience can be quoted in support of all-tar coating. Partridge recommends for severe conditions, as in the tropics, coating the bands with red lead; no corrosion found after 26 years.[21]

Design.‡ Staves are designed to take the unequal compression resulting from external loads. Swickard[44] develops the formula, based on compressive stress of 125 lb. per sq. in.,

$$t = \sqrt{0.0126WD + 9r^2} + 3r,$$

where t = stave thickness, in in.

W = external load, lb. per lin. in. of pipe., *e.g.*, for 6-ft. fill at 100 lb. per cu. ft. over an 8-in. pipe.

$$= \left(\frac{6 \times 100}{12} \times \frac{8}{12}\right) = 33.$$

r = rod radius, in in.

For smaller pipes, this formula gives t impractically small. Rods are designed to take internal pressure and not to crush the wood; the latter governs on small pipes.

(*a*) Band spacing, L_m, when metal strength governs: $L_m = \dfrac{S}{(PR + At)}$.

(*b*) Band spacing, L_w, when wood crushing governs: $L_w = \dfrac{(R + t)I}{(PR + At)}$.

where S = working strength of metal ring, in lb.

P = water pressure, in lb. per sq. in.

R = inside radius of pipe, in in.

t = stave thickness, in in.

A = unit compressive stress in staves, varying in a straight line from 125 to 15 for heads varying from 400 to 10 ft., respectively.

I = pressure, in lb. per lin. in. of band, on indented strip under band.

* Creosoted culverts installed in 1892 on Southern Pacific Ry., were free from decay in 1920.[44]

† See also *E. N. R.*, Vol. 79, 1917, p. 317. Bensen reports that tastes from standard creosoted pipe is not noticed after 26 days. In Tacoma pipes were treated with a preparation of cresylic acid and fuel oil; tastes were recurrent over a longer period.

‡ See also Parker, "Control of Water" (Van Nostrand, 1915), p. 465; and diagrams in *E. N. R.*, Sept. 4, 1919, p. 472.

Bands for continuous pipe are designed for tension; upsets should have areas at roots of threads sufficient to develop strength of rods. In machine-made pipe, a small wire or band closely spaced gives a more satisfactory pipe than a large metal element widely spaced; there is an economical limit, however (see also p. 364).

Specifications. Those proposed by Partridge in *T. A. S. C. E.*, Vol. 82, 1918, p. 459, should be used with caution, as brought out in the discussions.* See also Swickard in *E. C.*, Feb. 17, 1915, p. 146, and Ledoux in *J. A. W. W. A.*, Vol. 9, 1922, p. 567. Discussions contain excerpts from specifications of U. S. Reclamation Service and New York City Bureau of Sewers.

Conduit Details. Air valves should be installed at all summits and blow-offs at all depressions. The collapse of the Seattle pipe[22] was laid to ice blocking the vents. Air accumulations also hasten decay.[2c] Most makers of wood pipe sell air valves. Water hammer must also be considered (see p. 781, and *J. N. E. W. W. A.*, Vol. 9, 1922, p. 805). Economy will result if location requires fairly sharp curves and long tangents rather than long curves and short tangents. The flexibility of the pipe renders suspension bridges economical for deep, narrow valleys (see *E. N. R.*, Jan. 24, 1924, p. 167).

Costs. See *Bull.* 155, U. S. Dept. of Agriculture, 1915. Astoria pipe, 7½ mi. of 18-in., cost the city, complete, 90 cts. per ft. in 1895; including engineering, it cost about $2 per ft. to replace this line in 1911. At Jerome, Idaho, 3100 ft. of 40-in. pipe for 100-ft. head, in trench with 2-ft. cover, cost $2.87 in 1912 without engineering or administration charges; costs are analyzed by Swickard in *E. C.*, Feb. 17, 1915, p. 146; see also Table 94, p. 362.

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*** Many of those participating were commercially connected with the wood-pipe industry.**

CHAPTER 17

CONCRETE AND REINFORCED-CONCRETE PIPES*

Concrete Pipes† with inserted joints‡ are suitable for gravity water lines or for lines under slight pressure where leakage is immaterial. The best mixture for small pipes is cement 1:4 sand and gravel. For 20-in. and larger pipe, use 1:3 mix. To all batches add hydrated lime, 5 per cent. by weight of cement. Cast on end. Every inch of circumference should be carefully tamped; do not displace core when so doing; otherwise pipe will vary in thickness. Pipes even of best materials and workmanship may be ruined if not properly cured. Curing should take place in open air unless there is danger of freezing. After pipe has set for about 30 min. sprinkle by hand with fine sprinkler. Drench after 2 or 3 hr., and wet often enough after that to keep from drying out, for 1 week. After 7 days' curing, coat the pipes with a thin mixture of cement and water, pipes 4- to 12-in. diam. being dipped. All pipes should cure for 3 weeks before being placed in the trench. Cement at pipe joints should be allowed to set 1 hr. before refilling the trench; use only earth free from stones and tamp lightly around joint. When cover of 1 ft. above joint has been put in, ordinary refilling may be used; 36 hr., or preferably 48, should be given joints to set before turning water into the pipes.[1] See Standard specifications C-14-20, A. S. T. M. Unfortunately manufacturers have not standardized types of joint for interchangeability.

Concrete pipe has been used on many irrigation and water-supply projects in the West, and many machines have been developed for its manufacture, typical of which is the McCracken packerhead machine.‖ (For discussion of machines, see Lambie in *Proc. Eng. Soc. of Western Pa.*, Vol. 38, 1923, p. 486).

Failures have been not infrequent; chief causes are excessive pressures, unequal settlement, contraction. Differential expansion of the shell caused longitudinal cracking of a 20-in. pipe near Tucson.[2] Sewer gases have also been held accountable for disintegration of plain concrete pipe.[3] The Dept. of Eng., State of California, investigated pipes in 1920; recommendations are summarized in *E. C.*, Aug. 10, 1921, p. 143. For failure in alkali soils see p. 375.

Vitrified pipes of double strength have been used as sewer force mains under 7½-lb. pressure (12-in. diam.) at Shreveport, La. Asphaltic joint filler was used. Sound pipes are essential for this service.[4] An 18-in. pipe under 5-lb. pressure for water supply on Staten Island leaked excessively.[5] An

* Mixtures designated in this chapter are generally approximate only.

† Also called "cement pipe" and "concrete tile." Makers include Concrete Products Co., Pittsburgh, Pa.; Independent Concrete Pipe Co., Newark; Lock Joint Pipe Co.; Massey Concrete Products Co.; Pierce Mfg. Co., Bridgeport, Conn.; Twining Concrete Co., Fresno, Cal.

‡ Bell-and-spigot or slip-joint type.

‖ McCracken Machinery Co., Sioux City, Iowa.

18-in. fire-clay pipe on grade of 0.29 per cent. forms part of conduit for Ayr, Scotland, conveying 2.6 mgd.[6] Roots caused some troubles in joints of the 15-mi. conduits (6-, 7-, and 8-in. pipe) at Flagstaff, Ariz.[41] Infiltration into a tile line is claimed to have caused a typhoid epidemic at Brigham City, Utah.[42] Tight joints are essential.

Eternit.* In difficult isolated regions in Italy, pipes of "Eternit" have been used—a concrete made of asbestos fiber and Portland cement, remarkable for its lightness, hardness, strength, and imperviousness. It may have a compressive strength of 12,000 to 14,000 lb. per sq. in. and a tensile strength of 1,400 to 2,000 lb. per sq. in. Pipes of Eternit are impervious to heads of 400 ft. The Apulian aqueduct has an Eternit pipe line near Locorotondo, which is 6 in. in diam., 4920 ft. long, under hydrostatic head of 130 ft. There is also one in the Brindisi line, 12 in. in diam., 4265 ft. long, with hydrostatic head of 130 ft. They have given satisfaction. Promising experiments were made on their use for small diameters for distribution lines. These pipes can be sawed, turned, perforated, and riveted, and it is very likely that they may be adopted for use on service lines. Three types of joints are made.[7]

Cement-lined Pipe. For nearly 50 years water pipes have been lined with cement in New England, and to a less extent elsewhere, to counteract corrosion and tuberculation of metal (see p. 388). Plymouth has laid nothing else since 1855, the pressures varying from 30 to 70 lb.; in 1900 a shop was equipped for lining soft-steel screw pipes according to the Phipps patent.[8] The patent is controlled by American Pipe & Construction Co. Special tools are made by Union Water Meter Co., Worcester, Mass. The process of cement lining is not patented.[9] Heretofore, cement has been applied only to wrought-iron or steel pipes. For a list of notable installations outside of New England see Gibson, *E. N. R.*, Sept. 7, 1922, p. 387. In Saratoga wrought-iron pipe coated outside and lined with cement mortar was laid in 1845. Cement-lined cast-iron pipe has been in use in many cities since 1920. Cement lining is used at Waltham, Mass., and elsewhere to protect service pipes against corrosion by free CO_2. Stewarts & Lloyd, Ltd., Glasgow, have a process for lining centrifugally.

REINFORCED-CONCRETE CONDUITS†

Types. Conduits to carry water under pressure may be monolithic or of precast pipe; most of the latter types are patented. For heads above 100 ft., American practice previous to 1915 favored monolithic construction. Pressure pipes,‡ long used in Europe, were introduced about 1915.

Essentials. The conduit should have a wall thick enough to provide watertightness, rigidity, protection against erosion and corrosion; an interior surface smooth enough for good flow conditions, and sufficient reinforcement for the bursting, temperature, and handling stresses. Joints between pipes should require no bell holes, be watertight, not perfectly rigid, easily made in the trench, and capable of expansion. Good foundations and thorough backfilling are important.

* Italian agents are: Societá Anonima "Eternit," via Caffaro, Genoa.

† Standard specifications for Culvert Pipe are in preparation; address American Concrete Inst., Detroit.

‡ Also termed "steel-cylinder pipe," see p. 376.

Advantages. Possibility of construction by local labor; sustained carrying capacity; and durability. Use of local materials will cut costs. For comparison with wood pipe, see 358. High carrying capacity; $C_H = 138\text{–}152$ on Tulsa conduit (see *E. N. R.*, May 28, 1925, p. 894); $C_H = 146.5$ on Denver conduit (see *E. N. R.*, Apr. 29, 1926, p. 678.

Disadvantages. Deterioration in alkali soils, rusting of reinforcing and leakage resulting from poor construction. Difficulty of connections. Failure of a monolithic conduit under external loads occurred at Montreal. Section, 7½ by 9 ft. wide, designed for hydraulic grade 10 ft. above top of pipe, with earth cover of 5 ft. above crown, failed when canal was being dug 17 ft. away. Drag scraper on top of conduit imposed heavy loads, which, combined with inadequacy of reinforcement to take this unnatural loading, produced failure.[38] Damage of concrete pipe by electrolysis is not unknown; see Tech. Paper 52, 1924, U. S. Bureau of Standards.

Precast Pipe Conduit vs. Monolithic Conduit. No interruption during construction while invert forms are being placed and removed. Backfilling can follow immediately whereas on a monolithic conduit, it must await the setting of concrete. Forms constitute a less percentage of the cost of pipe. There is less waste of aggregate and cement at the centrally located plant for precast pipe. Methods of manufacture should result in more dense pipe. Trench opened is shorter with pipe. Pipe can be laid to long radius curves. On the other hand monolithic construction is not patented, and it may prove cheaper for heads below 15 ft., or for gravity lines.

Durability.* Alkali soils contain sulfates of magnesium, sodium, and calcium, which in the presence of ground water attack poor concrete from outside and transform it into a sulfate of calcium, destroying its cohesion. This has wrecked sewers in Winnipeg and St. Boniface,[10] and has affected concrete linings of some irrigation canals. On Winnipeg aqueduct, a portion of 96-in. pipe with 8-in. walls suffered decay; it was combated by providing underdrainage, so that the ground waters would not stand in contact with the pipe. If water contains more than 3500 p.p.m. alkali salts, studies in Minnesota indicate decomposition.[11]

Rusting. In Jersey City reinforced-concrete conduit tuberculation of steel has occurred where too near the surface of the concrete, like tuberculation on inner surfaces of steel pipe.

Carrying Capacity† in well constructed conduits is high but may, in some cases, be lessened by algae growths,‡ and by disintegration of the surface. Some elements leached out of the Sooke River conduit, and the interior, which Hazen described as "smooth as glass," became quite rough.‖ Slime may be expected from waters of reservoirs containing algae growths, especially in the East and Northwest. A depreciation of 10 to 15 per cent. can occur within 5 to 7 months, after which conditions remain fixed. Cleanings must be frequent to maintain original capacity; it may prove economical to design on basis of no cleanings.

* See particularly *Report* National Research Council, "Marine Structures, Their Deterioration and Preservation," 1924.
† See also p. 374.
‡ See p. 297.
‖ The pipe makers state that this has not occurred in other localities.

Table 97. Values of C_H and Kutter's "n" for Concrete-lined Pipes Flowing Full*[13]

Velocity, ft. per sec.	12 in. pipe		24 in. pipe		48 in. pipe		96 in. pipe	
	C_H	n	C_H	n	C_H	n	C_H	n
1	135	0.0106	137	0.0107	141	0.0114	150	0.0115
3	124	0.0110	127	0.0115	130	0.0119	136	0.0120
5	118	0.0110	124	0.0115	125	0.0120	130	0.0121
10	112	0.0111	116	0.0117	119	0.0120	123	0.0123

* Applies also to concrete and reinforced-concrete pipes.

Mixtures used by U. S. Reclamation Service vary from 1:1½:3 to 1:2:4. For important work under high heads, the Lock Joint Pipe Co. recommends reground sand;[39] and a mix of 1:1½:2½. Aggregate at St. John, N. B.,[25] was too "harsh" for watertightness; the addition of 2 per cent. clay resulted in a "fat" mix, flowing easily. Watertightness is best obtained by use of sufficient cement.

PRECAST PIPE CONDUITS

Reinforced-concrete pipe for pressures up to 150 lbs. per sq. in.* has been in successful use in Europe for more than 30 years, and in America for 15 years. It is being continually improved. The pipe may be (*a*) poured pipe, in which the concrete functions as a water stop, to protect the reinforcing steel and to provide a smooth waterway; (*b*) steel cylinder pipe (Bonna pipe)

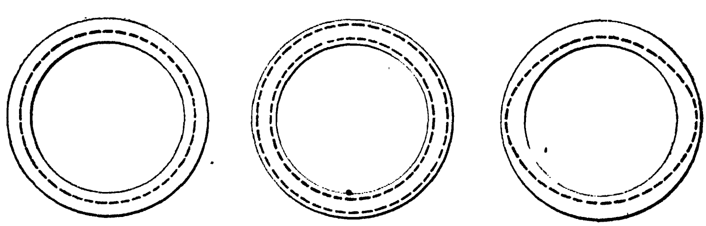

Fig. 168.—Reinforced concrete pipe. Three positions for mesh reinforcement: *a*, One layer to resist internal pressure; *b*, two layers for great depths or high internal pressure; *c*, one layer to resist external pressure.

in which the concrete functions to protect the welded steel cylinder, which is the water stop, and the reinforcing steel, and to provide a smooth waterway; (*c*) centrifugal pipe, in which the concrete functions the same as in (*a*) but is more dense, and therefore capable of higher heads.

Typical installations. Recent lines using lock-joint pipe are: East Orange, N. J.,[24] 20-in.; Greeley, Col.,[24] 27-in.; Norfolk, Va., 36-in.;[12] Tulsa, Okla.,[19] 54- and 60-in.; Denver, Colo.,[20] 54-in.; Cumberland, Md.,[21] and Winnipeg.[22]

Steel-cylinder pipe† is recommended where the head exceeds 100 ft. Over 100 mi. of pipe containing a thin sheet-steel waterstop with spiral reinforcement for structural strength were installed in vicinity of Paris before 1900; heads, up to 130 ft.; diam., 1 to 6.5 ft.[17] A 20-in. line at Swansea, Wales,

* For use on low-head pipes, see *E. N.*, Sept. 28, 1916, p. 592, and *E. R.*, Jan. 27, 1917, p. 136.

† Also termed "pressure pipe."

installed in 1905, and tested under 450-ft. head without leaking, is still in service. Bonna system is widely used in Europe.* Lock-Joint Cylinder pipe was recently used for the three high-service lines, Washington, D. C. Corrosion of steel cylinder is little feared, due to deoxidized condition of the water which may come in contact with it through an accidental crack.

Table 98. Reinforced-concrete Conduits with Sheet-metal Core in Great Britain[18]

Location	Date of laying	Use of main	Diameter, in.	Working pressure, ft.
Swansea	1905–1906	Water	20	246
Norwich	1908	Sewage	36	131
Swansea	1910	Water	24	500
Clydebank	1910	Water	18	390
Birmingham	1921	Water	60	255

Centrifugal pipe, the latest type, is rendered extra dense by whirling at high speed in molds; Lock Joint Pipe Co., (1924) makes sizes from

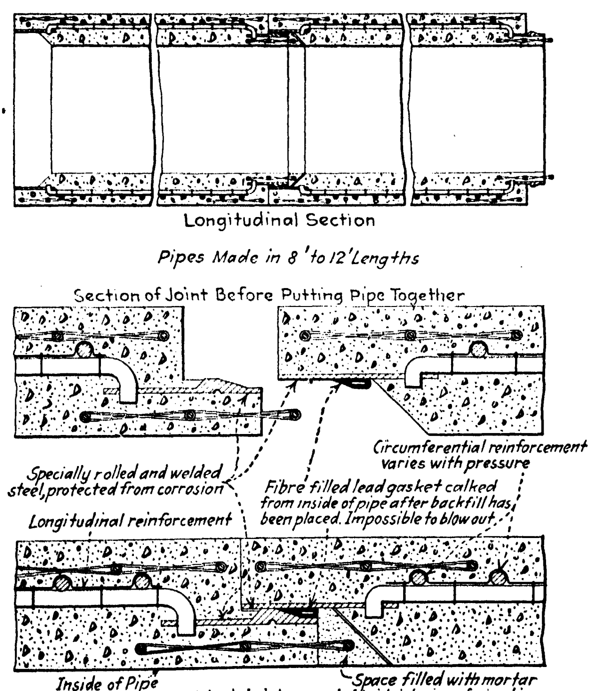

FIG. 169.—Lead and steel joint.
(Lock Joint Pipe Co.)

12- to 36-in. It is also made in United States by Centrifugal Concrete Products Co. of America, Dallas. Centrifugal methods have been popular in Australia, South Africa,[23] and Italy[7] for a number of years. For East Orange waterworks 20-in. pipe was made in 1922 and for Greeley, Colo., 27-in. in 1922.[24] For bursting tests, see *E. N. R.*, Oct. 9, 1924, p. 595.

* For use at Antwerp, see *E. C.*, July 31, 1918, p. 117.

Contraction Joints[14] are necessary, as a range of temperature of 35 degrees from summer to winter is common in covered pipe. In uncovered pipes, it would be much greater. Cracks and leakage result where contraction joints are omitted.* The early Sooke river line[27] which has "sewer pipe" joints, and no contraction joints, contrary to the advice of the pipe company, leaked heavily for several years (see test data in *E. N. R.*, June 17, 1920, p. 1210). Improvements to joints in recent years have decreased leakage. The two types recommended by the Lock Joint Pipe Co. are shown in Figs. 169 and 170. They can be used in any of the 3 types of Lock Joint Pipes. No bell holes are required; a lead gasket provides tightness; they can be deflected up to 6° or 7° without leaking.

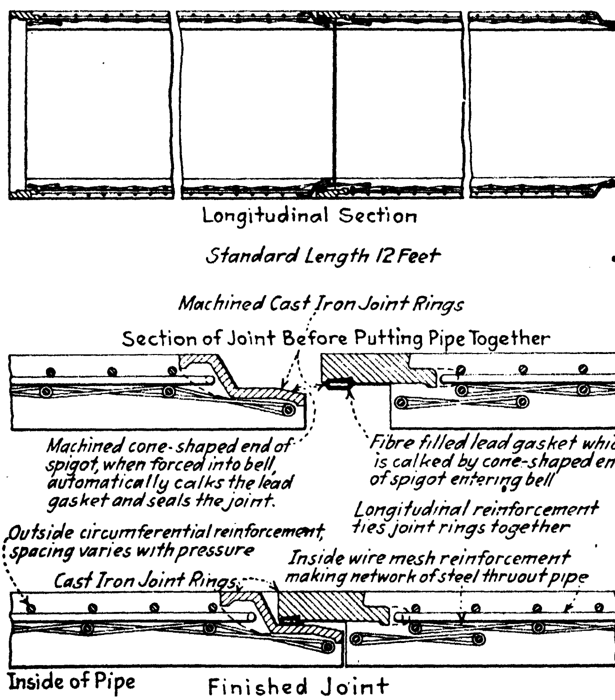

FIG. 170.—Lead and iron joint.
(Lock Joint Pipe Co.)

Leakage. Contract requirements generally limit leakage to 200 or 250 inch gals. per mi. per day. For data on typical installations, see *J. A. W. W. A.*, Vol. 5, 1918, p. 423, and *J. N. E. W. W. A.*, Vol. 38, 1924, p. 250 et seq.

MONOLITHIC CONDUITS

Monolithic conduits are used where the heads are too great for precast pipes. Notable aqueducts are: Kensico by-pass, Catskill aqueduct; Jersey City;[29] Cedar Grove, N. J.;[30] Huesca, Spain;[31] Pinto and Cottonwood, Ariz.;[32] Forest Boise, Idaho;[33] Saugus, Los Angeles;[34] Palazzo San Gervasio, Italy;[7] Grenoble, France.[28]

* See Longley, *J. N. E. W. W. A.*, Vol. 38, 1924, p. 258.

Design Methods. F. F. Moore, in *J. Assn. Eng. Soc.*, Vol. 47, July, 1911, outlines designing methods for reinforced portions of Catskill aqueduct. For large diameters, not far below hydraulic gradient, loads are not uniform around the circumference. Internal load results from hydrostatic head, generally about two to three times the diameter of pipe, giving greater pressure at bottom than at top. External load comprises weight of masonry, refill, and embankment. Non-uniform loading induces in ribs, besides direct stresses, flexural stresses of important magnitude. Sections were investigated for controlling load conditions at all critical planes with reference to both stresses; in general, design of steel to take all stress met condition imposed for concrete and steel acting together.

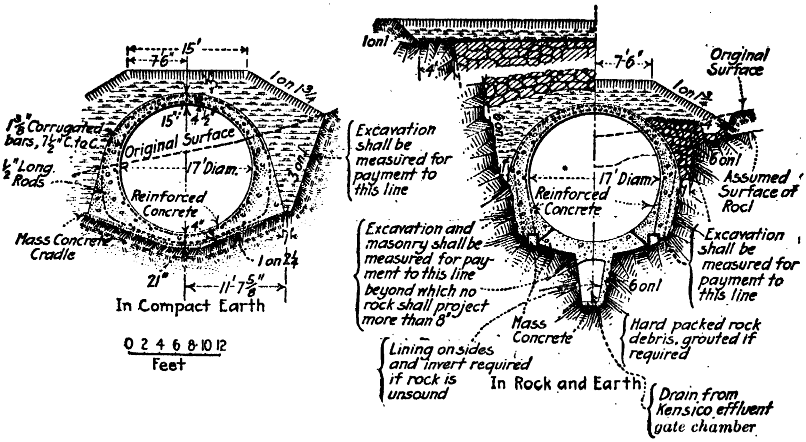

FIG. 171.—Reinforced concrete aqueduct. Types in open cut.
(Kensico effluent, Catskill aqueduct.)

Trial sections, perfected sufficiently to answer the purpose, were used to determine economic shape of masonry section, and relation between concrete and steel areas. This involved investigation of several possible loadings to discover critical position. Stress determinations were largely graphical.* Resultant force lines were drawn in usual way; that force line which fixes position of center of internal stress in materials being determined by Castigliano's theorem of least work.†

Semicircular conduit,‡[37] 24- to 36-in. diam., under maximum head of 23 ft., was built of gunite on mesh at Tacoma, Wash. Top was a 3-in. precast flat slab. This shape was selected as best adapted to application of gunite.

Palazzo San Gervasio conduit[7] (Fig. 172) has series of ribs supporting the pipe; they are designed to take care of secondary stresses, which are important in pipes of large diam., causing a shifting of the tension curve some distance out. Some steel is placed as far out as possible. This siphon is 14,300 ft. long, and under 79-ft. head.

* The usual preliminaries of arch analysis were followed, *i.e.*, the arch rib was divided into voussoirs of equal length along the axis, the magnitude and direction of forces determined, etc.
† Burr, "Elasticity and Resistance of Materials of Engineering," (Wiley, 7th ed.), p. 788, 1914.
‡ See also p. 278.

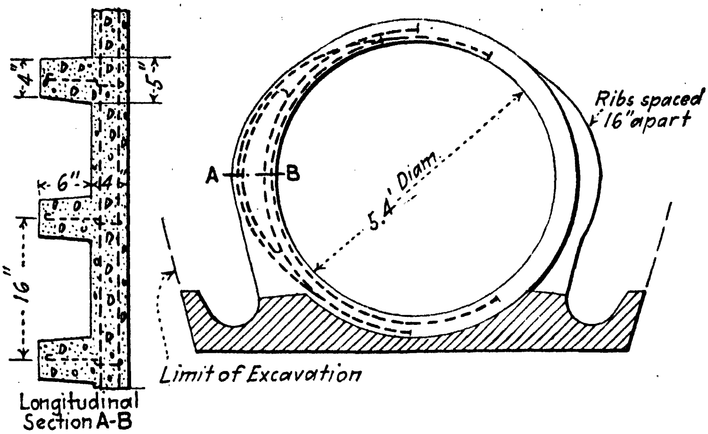

FIG. 172.—Reinforced-concrete ribbed siphon.

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PART IV

DISTRIBUTION OF WATER

CHAPTER 18

CAST-IRON PIPE AND FITTINGS

Use. Water is distributed in most American municipalities through cast-iron pipes. Connections between the street mains and the houses, known as "service pipes," are cement-lined, brass, or wrought pipe (see pp. 349, 374, 431).

Advantages of cast-iron pipe: Thickness sufficient to provide ample safety against corrosion, bursting, handling, air pressures,* and trench loads; dimensions standardized; jointing by cheaper labor than in steel pipe; tapping and making service connections more readily done than with steel, wood,† or reinforced concrete; long life; good hydraulic flow condition, if properly coated and maintained;‡ dependability under fire demands. In Santa Barbara earthquake, 1924, the cast iron lines, 124 miles long, suffered but 2 breaks.

Disadvantages. First cost§; weight; high freight rates; tuberculation by soft and colored waters which may reduce carrying capacity as much as 71 per cent.‖; liability to electrolysis; external corrosion from acid soil. Under usual specifications, waste¶ of material may be possible. Steel pipe for special conditions, *e.g.*, dipping under sewers, can be fabricated to dimensions required; cast-iron pipe specials requiring a new pattern and mold would raise the cost beyond reason; use of standard specials would lower the cost but more space might be occupied.

Specifications. N. E. W. W. A. adopted standard schedules and specifications, Sept. 10, 1902. Subsequently other schedules and specifications were issued by A. S. T. M. (Nov. 15, 1904), U. S. Cast Iron Pipe & Foundry Co., American Cast Iron Pipe Co., American Foundrymen's Assn., and Canadian Soc. C. E., which are essentially like those adopted May 2, 1908, by A. W. W. A. and regarded as the manufacturers' standard. These two standards have few differences in the specifications, but the schedules and the castings made from them differ materially because of the methods of adjusting diams. to changes of thickness to obtain various classes of pipe for various pressures in service. Manufacturers, however, often make pipe on order differing from these standards.** For comparison of the United States stand-

* Elimination of air valve makes a conduit to that degree foolproof.
† See p. 367.
‡ See also p. 421, as to cleaning.
§ On basis of ultimate cost (maintenance and depreciation capitalized and added to the bid prices) Cleveland selected lock-bar pipe in 1922 for the 60-in. conduit, Kirtland station to Fairmount reservoir.
‖ Extreme value and for small pipe.
¶ Use of thicker pipe than pressures require.
** Rochester used 37-in. pipe at a saving on 7.7-mi. conduit (see *E. R.*, Sept. 25, 19[illegible]).

ard bell with the English* and French, see *J. A. W. W. A.*, Vol. 8, 1921, p. 162. Carson[1] claims that the American standards result in pipe 50 per cent. heavier than used in European practice.

Revision. Committees from the two waterworks associations have been collaborating since 1916 on revisions (see the *Journals* for Progress Reports). The important changes suggested are: uniform outside diam. for different classes (thicknesses) of pipes of the same nominal inside diam.; inclusion of flanged pipes; definite relation preserved between breaking load and flexure;† chemical composition; improved coatings. In the N. E. schedule there is an inconsistency which should be corrected: for 30-in. and larger pipes and specials, the thickness of the bell, or socket, is less than the thickness of the barrel of the pipe, for the heavier classes.

Design Standards. The N. E. and A. W. W. associations publish their standards in pamphlet form, for sale by the secretaries at nominal charge; catalogs of manufacturers also contain these data. They should be in the files of every waterworks designer. Manufacturers make many fittings‡ which can be utilized to meet special conditions. Such fittings include wall castings, offsets, bell mouths, side-outlet tees, side-outlet bends, headers, and reducers. See dimensions in catalogs, notably American Cast Iron Pipe Co., James B. Clow & Sons, Warren Foundry & Machine Co., R. D. Wood & Co., U. S. Cast Iron Pipe & Foundry Co. Pump manufacturers§ also utilize some short specials to save space. For important connections steel castings are frequently used, instead of cast-iron specials.

Thickness of Cast-iron Water Pipes.[2] The heads in ft. to which the cast-iron pipe thicknesses specified by N. E. W. W. A. conform, are given by the formula:

$$(H + H_r) = \frac{15{,}206.4(t-0.25)}{d}, \text{ or } (P + P_r) = \frac{6600\ (t-0.25)}{d}$$

H = head, ft.; P = pressure, lb. per sq. in.; t = thickness, in.; d = diam., in.; H_r and P_r being allowances for water-hammer. Committee of A. W. W. A. recommends the following values for H_r and P_r:

d, In.	H_r, Ft	P_r, Lb Per Sq In.
4, 6, 8, 10	277	120
12 and 14	254	110
16 and 18	231	100
20	208	90
24	197	85
30	185	80
36	174	75
42 to 60	162	70

Fanning's formula[2] for thickness in in , t:

$$t = \frac{(P + 100)d}{0.4S} + 0.333\left(1 - \frac{d}{100}\right)$$

S = ultimate tensile strength of cast iron, lb. per sq. in.

* British Eng Standards Comm , issued specifications in 1917.
† Dr. Moldenke investigated this for the manufacturers. See views in *J. N. E. W. W. A.*, Vol 37, 1923, p 1.
‡ For important connections, steel castings are sometimes used.
§ Grinnell Co. have developed specials.

One of the heaviest cast-iron pipes known is at Scranton, Pa., where there are 3 mi. of 48-in. pipe, parts of which are under 600-ft. head; the pipes are 2 in. thick. It has been found by experience that, for handling over rough roads, 30-in. cast-iron pipe should not be less than $\frac{7}{8}$ in. thick.[3] Thicknesses required in the standard schedules have been often questioned as wasteful of materials. Diven[4] cites a pipe at Troy holding for 85 years, which in a portion of its eccentric section was but 25 per cent. of thickness now required. He reports a class B pipe operated successfully under 165-lb. pressure. J. N. Chester[5a] believes the present weights too heavy; American Waterworks & Electric Co. successfully used 6-in. 30-lb. pipe under 300-lb. pressure at South Pittsburgh. Metcalf knows of miles of 6- to 12-in. class A pipe that have stood 125 lb. for decades[5b] (see also p. 400). At Whittier, Cal. in 1921, use of iron tested to 23,000 lb. per sq. in. and proportioning pipe thickness by Fanning's formula saved on 305 tons, $13,000 on manufacturing and $20,000 in freight.[6] High-strength* pipe has been installed also at Greenville, S. C., Omaha, Tulsa, Kansas City, Mo.

Foundry Inspection. Cast-iron pipe should be inspected at the foundry by inspectors delegated by the engineer. Particular attention should be given to specials. Crosses are more often defective than tees;[7] the failure of a cross puts the four intersecting lines out of service. Need of rigid inspection is shown by Saville's experience on Hartford waterworks[8] (1912–1915); 11 per cent. of 6588 tons of straight pipe inspected was rejected; sizes, 4 to 24 in., class C pipe; rejection of specials averaged 24 per cent. The chief causes of rejection were: bad bead, dirty bell, core scabs, and uneven pipe.

Methods of manufacture and inspection are given by W. R. Conard, *J. N. E. W. W. A.*, Vol. 35, 1921, p. 205; by T. H. Wiggin, *J. Assn. Eng. Soc.*, Vol. 22, 1899, p. 218, and Maury, *J. A. W. W. A.*, Vol. 12, 1924, p. 7; the same for Duane type of flexible joint, in *E. N. R.*, May 11, 1922, p. 780.

Stresses Due to Refill.[9a] Forces considered: internal static pressure, water-hammer, those caused by earth refill over and around pipe.

d = inside diam., in.
t = thickness, in.
F = depth of earth above top of pipe, ft.
s = permissible stress, lb. per sq. in.; 4400 for cast iron (equivalent to safety factor of 5 for good iron).
W = weight of earth over 1 lin. in. of pipe, assume 115 lb. per cu. ft., and outside diam. of pipe $1.05d$; $W = 0.84Fd$.
d_1 = average diameter of shell = about $1.025d$.
M = breaking moment normally present from earth
$= \frac{1}{16}Wd_1 = 0.0538Fd^2$.

Resulting maximum circumferential stress in metal is

$S_1 = 6M \div t^2 = 0.323Fd^2 \div t^2$; stress available for resisting water pressure is 4400 less this amount; of which one-third is allowed for water ram. Hence stress allowable for resisting static pressure is:

$$S_2 = \tfrac{2}{3}(4400 - 0.323Fd^2 \div t^2).$$

* Trade name is "Hi-tensile."

H, head in ft. that can be carried by a given stress,

$$= (13{,}500t \div d) - Fd \div t.$$

Allowing 0.10 in. in country pipes and 0.25 in city, for inaccuracies of manufacture, etc.

$$t = \frac{d}{27{,}000}(H + \sqrt{54{,}000F + H^2}) + 0.10 \text{ (country)}$$

$$t = \frac{d}{27{,}000}(H + \sqrt{54{,}000F + H^2}) + 0.25 \text{ (city)}$$

General formula:

$$t = \frac{d}{9.24S}(H + \sqrt{27.5FS + H^2}) + 0.10 \text{ (country)}$$

$$t = \frac{d}{9.24S}(H + \sqrt{27.5FS + H^2}) + 0.25 \text{ (city)}$$

Assuming that pressure due to refill is vertical and uniformly distributed over upper surface of pipe, with no considerable horizontal pressures against pipe, elastic flattening y, in in., or reduction of vertical diam. of relatively thin pipe is:

$y = w(d + t)^4 \div 8Et^3$, whence $w = 8Et^3y \div (d + t)^4$.
w = uniformly distributed vertical load per sq. in., lb.
E = modulus of elasticity of pipe metal = 11,000,000 for cast iron and 30,000,000 for steel and wrought iron.

Data from experience or experiment are few, but assume, where heavy road rollers, etc., do not pass over pipe, that at least two-thirds weight of refill acts as uniformly distributed vertical load on pipe and produces circumferential bending stresses. Whence maximum stress in outermost fiber, $S = 0.375w(d + t)^2 \div t^2$. Assuming refill 6 ft. deep at 115 lb. per cu. ft., $S = 1.2\left(\frac{d+t}{t}\right)^2$. In general, $(d + t) \div t = 37$ for thinnest large cast-iron pipes and 193 for thinnest large steel pipes. Severe circumferential stresses are caused in cast-iron pipes by settling of several lengths, producing leverage pressures in joints on tops and bottoms of spigots. Assume this load vertical, uniformly distributed over horizontal mean diam. and on effective width of 2 in., and resisted by 12 linear in. of spigot end of pipe; then unit load,

$$w = 16s\left(\frac{t}{d+t}\right)^2.$$

Taking 27,000 lb. per sq. in. as ultimate strength for cast iron in flexure, for 36-in. pipe, 1 in. thick, w = 316 lb. per sq. in.[9b] See also *Bull.* 22, Univ. of Illinois Eng. Expt. Sta., A. N. Talbot, 1908; *E. N.*, Dec. 15, 1904, p. 547, W. W. Patch; *T. A. S. C. E.*, Vol. 38, 1897, p. 93, D. D. Clarke.

Design of Pipe. By formula developed at the Iowa State College Experiment Station:[41]

$W = cwb^2$, where W = total load per lin. ft. of pipe, c = coefficient taken from following table, w = estimated weight of back-filling material, lbs. per cu. ft., and b = breadth of trench at or slightly below top of pipe, h = depth in feet of backfilling over top of pipe.

Table 99. Values of Coefficient *c*

Ratio $\frac{h}{b}$	Min for unsettled common soil	Maximum for sand	Wet clay	Maximum for saturated clay
4	2 04	2 22	2.49	2 66
5	2 22	2 45	2 80	3 03
6	2 34	2 61	3 04	3.33
7	2 42	2 73	3.22	3.57
8	2 48	2 82	3 37	3.76
9	2 52	2 88	3 48	3 92
10	2 54	2 92	3 56	4 04

Coefficient c for wet clay is recommended for use in all cases except where entirely different material is known to exist. The materials named in the table are estimated to weigh 100, 120, 120, and 130 lb. per cu. ft., respectively in the order named.

Breakages of cast-iron pipes, although but a very small percentage of total number of pipes laid, are, unfortunately, frequent and sometimes disastrous. Common causes are: unequal bearing (a rock or other unyielding object under a portion of the pipe); excessive external load, due to refilling in trench; undiscovered defects of casting; cracks caused in transportation or laying. An unusual cause is assigned by John W. Alvord for breaks in Cincinnati, Ohio, one in a 60-in. main, $1\frac{5}{8}$ in. thick, in good condition, bottom 15.5 ft. below street surface, Dec. 13, 1913; the other in a 48-in. main, Sept. 13, 1914. At and near breaks, each spigot, at some place in its circumference, bore on bottom of adjacent bell, and pipe line was laid on a horizontal curve of large radius. External pressures due to moving ground and other causes were exerted on convex side of curve, throwing pipe into longitudinal compression. In each case a large piece was broken out of one pipe, including the half of the bell on the convex side of the curve, and tapering toward the spigot, being 8 ft. long in the 60-in. pipe and over 10 ft. in the 48-in.[10]

In 20 years (1894–1914) there have been 47 breaks in the 48-in. main of Louisville Water Co.[11] from Crescent Hill reservoir, chiefly in cold weather. Pressures have ranged from 10 to 85 lb., but most of the breaks appeared independent of pressure; 21 breaks were circumferential (cracks), 25 were either longitudinal splits, or had pieces broken out, and 1 was a split hub. Leisen concludes that temperature stresses—both latent internal, and cold-weather stresses—were chief cause. The special type of joint prevented pipe accommodating itself to settlements (Hupe).

In 22 years there have been[12] 67 breaks in Hartwell Ave. line, Philadelphia, and in 10 years, no breaks in the similar line on Rex Ave. Davis blames poor quality of metal; high phosphorus content—1.33 per cent. The firm supplying this pipe has discontinued pipe founding.[43]

Reinforcing Weak Castings. In obtaining an emergency supply for Worcester, Mass.,[13] September, 1911, no time was available for returning to the pipe foundry 30-in. pipe castings not up to the specified thickness. A local foundry shrunk on circular reinforcing bands of steel, giving a factor of safety of 3 or 4.

Cost of Cast-iron Pipe. Fittings cost about 175 per cent. of standard bell-and-spigot pipe; flanged pipe, 250 per cent., and flanged specials 250 per cent.

In rough estimates of cost of systems, it is usual to figure only on straight pipe, and apply a factor of 1.05 to 1.10 to cover specials. Records of fluctuations of costs of cast-iron pipe are often required in valuation suits. Consult: "The Price History of Cast-Iron Pipe" (U. S. Cast Iron Pipe & Foundry Co.);

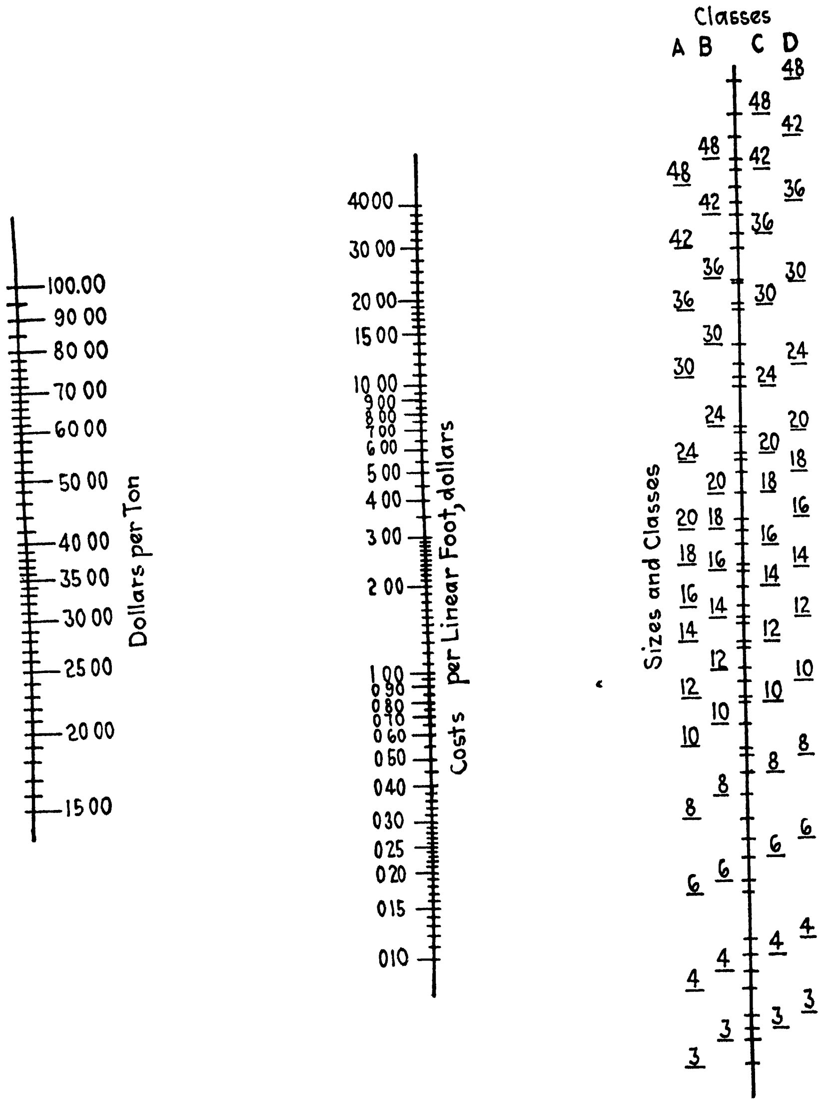


FIG. 173.—Diagram of costs of cast-iron water pipe.
(Based on diagram by W. E. Miller, *J. A. W. W. A.*, Vol. 1, 1914, p. 613. Pipe classes refer to A. W. W. A. standards.)

Fire and Water Eng., Jan. 10, 1923, p. 98, *J. A. W. W. A.*, Vol. 5, 1918, p. 148, and Vol. 11, 1924, p. 762; and *E. C.*, April, 1924, p. 789. Better prices are sometimes obtained by off-season purchasing; see *E. N. R.*, Oct. 26, 1922, pp.

675, 765; Gordon's study indicates that the summer purchaser has heretofore got better prices;[14] but Sherman questions this in "Statistical Analysis of Cast-Iron Prices," 1900–1923.*

Figure 173 is useful in making estimates as prices of cast-iron pipe are quoted by makers by the ton, while the engineer is generally interested in costs per lin. ft. Most manufacturers use A. W. W. A. specifications. Having class and cost per ton, as, say, 24 in. class D, and $60 per ton, lay a straightedge through these points on right and left lines respectively, and read $9.50 per lin. ft. on center line.

Capacity. For flow formulas, see pp. 754–760. All studies of capacity are conditioned on age of pipe, since corrosion† produces tubercles and roughnesses which cut down the discharge materially. Tubercles can be removed and carrying capacity improved by cleaning (see p. 421). The 10-inch conduit at Malone, N. Y.,[15] sustained its capacity for 38 years, and tested to $C_H = 130$.

Discussions of relative carrying capacities of cast-iron, wood, steel, and reinforced-concrete pipes in the engineering press should be interpreted in the light of the commercial connections of their authors.

Depreciation, or deterioration of carrying capacity, in a pipe protected from electrolysis is caused chiefly by corrosion, sedimentation, or incrustation on the interior, Hodgman concludes from replies to a questionnaire, summarized in *J. A. W. W. A.*, Vol. 5, 1918, p. 148. He cites a pipe at Lockport 20 years old which had lost 60 per cent. of its capacity.

Uncoated pipe conveyed Schuylkill River water in Philadelphia for 98 years; accumulation of scale measured ½ to ¾ in.[16] In 1897, 1000 ft. of 22-in. pipe laid in 1817 in Philadelphia was found tuberculated to only 4.2 per cent. loss of waterway.[24e] Deterioration of 25 per cent. in 8 years in the Liverpool 42.5-in. main is reported by Macaulay,[18] and of 40 per cent. in 20 years for Manchester, 44-in. mains.

Nodules consist largely of ferric and ferrous oxides and originate, apparently, from the action of CO_2 in the water on the metal through imperfections unavoidable in all forms of bituminous coatings (see analysis in *E. N.*, Aug. 6, 1914, p. 328). Iron bacteria are also an agency. Lime treatment and deaeration have been tried as preventives, and have proved unsuccessful and costly.[18] The following corrosion phenomena are cited by Chester: lime and iron coagulants tuberculate some pipes rapidly; hypochlorite applied without filtration, and acid mine waters both act as tuberculators. It was observed that 4-in. pipe lost capacity faster than 16-in. under the same conditions; eddy currents are held accountable for this.

Soil (or graphitic) corrosion‡ is not well understood. Often corrosion attributed to stray currents is due to soil; the confusion arises because both forms involve chemical action. Corrosive soils include those formed by vegetable decay, such as muck, peat, and others found in swampy places. Some alkaline soils of the Western United States and Canada are also destructive of iron and lead pipes.

Self-corrosion of iron pipes remote from sources of electrolysis has been reported from Selkirk and Brandon, Man.,[20] at Calgary, and at Pierre, S. D.,

* Harvard Graduate School of Business, 1924.
† In the case of soft waters.
‡ For electrolysis, see p. 428.

as occurring both on the barrel and at the joints. The soil is generally a wet alkaline clay in which Portland cement concrete deteriorates rapidly. Cinders, ashes, and filled ground containing refuse often produce extremely acute and rapid corrosion. Salt marshes not only corrode but accelerate stray-current electrolysis, and investigations by Davies[19] led him to conclude that graphitic corrosion is probably an electro-chemical reaction set up within the substance; that the less homogeneous the cast iron, the greater the graphitic corrosion; that in injurious waters gray cast iron will be acted on unless made in molds with burnt sand facings, and further protected by dipping in hot pitch or asphaltum.

Protection against corrosion is required both to prolong structural life of pipe, and to sustain its carrying capacity. Standard specifications for *coatings* have not produced the desired protection against tuberculation; committees of the waterworks associations are now promulgating new ones. Killam[21] cites the disappearance of coatings conforming to the specifications from pipes in storeyards of the Metropolitan Works, Boston. Coatings* are generally applied by dipping (see Wiggin, *J. Assn. Eng. Soc.*, Vol. 22, 1899, p. 222).

For coating very large special castings, modification of the specified requirements is necessary, since there are not large enough heating ovens and dipping vats in existence. A tar or asphalt varnish may be applied with a brush to the cold casting, after thorough cleaning, or somewhat better results may be obtained by locally heating the casting with large gas torches and immediately applying the varnish hot. Foundries today, in general, use a crude coal tar for coating; Church[22] believes that better results would be obtained if limits are fixed for consistence and free carbon in the tar. Spring Valley Water Co., San Francisco, dips its own pipe (see p. 318). Bitumastic enamel on cast-iron pipes of Narrows siphon, New York, was found in good condition, with little sign of tuberculation, after 7 years.[42]

Cement lining of cast-iron pipe is recent (for cement lining of wrought-iron and steel pipe, see pp. 324 and 374). Pipes at Charleston, S. C., could not be kept free of tuberculation, due to the action of the soft water, although repeatedly cleaned. Tests on a 6-in. line 35 years old showed Chezy's C = 34 before and 90 immediately after cleaning; 14 months later, C was 61. Soil conditions were unsuited to steel or wrought-iron pipe. Therefore, it was decided to install new pipe, cement—lined at the foundry; Gibson described methods in *E. N. R.*, Sept. 7, 1922, p. 389. A 42-in. cast-iron conduit supplying Liverpool from a distribution reservoir is being lined with a strong cement mortar applied by a special spinning process.[23] Cement-lined pipe is manufactured by all makers; they have proposed a Standard Specification.†

Life of Cast-iron Pipe. A section of 12-in. cast-iron service pipe, made by the Washington Iron Works, Newburgh, N. Y., laid in 1854, as a portion of the mains of that city, was dug up in 1909. The casting was originally $\frac{9}{16}$ in. thick, and was buried in ordinary meadow land; long service had produced very little reduction in section; the outside appeared in almost the same condition as when laid, having been protected by a deposit of alkaline scale;

* See pp. 314, and 814.

† It is the present practice (1926) of C. W. Sherman, to specify cement-lined cast-iron pipe for all new lines.

the inside showed some pitting and corrosive effects. Cast-iron pipes laid in Paris and Versailles, France, in the seventeenth century, are reported still in service and good condition after 260 yrs.

London and Glasgow have records of 120 years' service for cast-iron pipe.[24a] Pipes intended for 150-ft. head were used in Lowell, Mass., from 1830 to 1890 under 200-ft. head without signs of failure.[25] Examination in 1913 of 48-in. pipe laid in 1867 near the large reservoir in Central Park, New York City, in low-lying ground, wet most of the year, showed outside coating practically perfect.[24b] See also p. 387.

Bell (Hub)-and-spigot Pipe. Corrosion of flanged joints in system of Chelsea Water Co., London, led Simpson to introduce bell-and-spigot pipe in 1785.[26] Weakness of this joint is inability to take excessive pressures without blowing longitudinally. At curves where thrust is unbalanced by friction on the pipe, the bend must be secured to adjacent pipes by lugs and rods, or by masonry abutments (see p. 404).*

Laying length of cast-iron bell-and-spigot pipe is 12-ft. standard, but makers are prepared to make on order 16-ft. lengths. This saves weight, lead, calking labor, freight, and reduces possibility of joint leakage 25 per cent. Council Bluff, used the longer lengths in 1915 on a 6-in. line with a saving of 3 per cent. in cost.[28]

Flanged Pipe and Fittings. Standard schedule of flanged fittings and flanges was adopted Mar. 20, 1914, by Joint Committee of National Assn. of Master Steam and Hot Water Fitters, Am. Soc. M. E.,† and Com. of Manufacturers on Standardization of Fittings and Valves. Effective Jan. 1, 1915. Tables 100, 101, and 102 contain committee's schedules (which supersede all previous schedules).

Specifications proposed by *A. and N. E. W. W. A.*[29] Flanges shall be cast solid and shall be accurately faced smooth and true. Holes for bolts or studs shall be drilled, and the flanges shall be tapped where required. The contractor shall furnish and deliver all bolts and nuts for bolting on manhole covers. The bolts and nuts shall be of the best-quality wrought iron or mild steel, with good, sound, well-fitting threads, the nuts to be cold punched. The heads and nuts shall be hexagonal and shall be trimmed and chamfered. The heads, nuts, and threads shall be of the U. S. Standard sizes.

Flanged joints vs. bell-and-spigot. Flanged joints cost at least 80 per cent. more, are tighter under high pressures, have almost no flexibility to take up irregularities in alinement, will not blow out at curves, are easier to replace single pieces, and will not leak when subject to pump vibrations.

* Blowing out of a 45° bend in a 42-in. main at Wilmington, Del., is described in *E. R.*, Sept. 25, 1915, p. 390.

† Committee is preparing revisions (1926).

Table 100. Standard Flanges*
(All dimensions in inches)

Pipe size, in.	Standard weight. Working pressure, 125 lbs. per sq. in.					Extra heavy. Working pressure, 250 lbs. per sq. in.				
	Flanges		Bolts†			Flanges		Bolts†		
	Diam. D	Thickness T	Diam. of bolt circle C	Number	Diameter	Diam. D	Thickness T	Diam. of bolt circle C	Number	Diameter
1	4	7/16	3	4	7/16	4½	11/16	3¼	4	5/8
1¼	4½	1/2	3⅜	4	7/16	5	3/4	3¾	4	5/8
1½	5	9/16	3⅞	4	1/2	6	13/16	4½	4	3/4
2	6	5/8	4¾	4	5/8	6½	7/8	5	4	5/8
2½	7	11/16	5½	4	5/8	7½	1	5⅞	4	3/4
3	7½	3/4	6	4	5/8	8¼	1⅛	6⅝	8	5/8
3½	8½	13/16	7	4	5/8	9	1 3/16	7¼	8	3/4
4	9	15/16	7½	8	5/8	10	1¼	7⅞	8	3/4
4½	9¼	15/16	7¾	8	3/4	10½	1 5/16	8½	8	3/4
5	10	15/16	8½	8	3/4	11	1⅜	9¼	8	3/4
6	11	1	9½	8	3/4	12½	1 7/16	10⅝	12	3/4
7	12½	1 1/16	10¾	8	3/4	14	1½	11⅞	12	7/8
8	13½	1⅛	11¾	8	3/4	15	1⅝	13	12	7/8
9	15	1⅛	13¼	12	3/4	16¼	1¾	14	12	1
10	16	1 3/16	14¼	12	7/8	17½	1⅞	15¼	16	1
12	19	1¼	17	12	7/8	20½	2	17¾	16	1⅛
14	21	1⅜	18¾	12	1	23	2⅛	20¼	20	1⅛
15	22¼	1⅜	20	16	1	24½	2 3/16	21½	20	1¼
16	23½	1 7/16	21¼	16	1	25½	2¼	22½	20	1¼
18	25	1 9/16	22¾	16	1⅛	28	2⅜	24¾	24	1¼
20	27½	1 11/16	25	20	1⅛	30½	2½	27	24	1⅜
22	29½	1 13/16	27¼	20	1¼	33	2⅝	29½	24	1½
24	32	1⅞	29½	20	1¼	36	2¾	32	24	1⅝
26	34¼	2	31¾	24	1¼	38¼	2 13/16	34½	28	1⅝
28	36½	2 1/16	34	28	1¼	40¾	2 15/16	37	28	1⅝
30	38¾	2⅛	36	28	1⅜	43	3	39¼	28	1¾
32	41¾	2¼	38½	28	1½	45¼	3⅛	41½	28	1⅞
34	43¾	2 5/16	40½	32	1½	47½	3¼	43½	28	1⅞
36	46	2⅜	42¾	32	1½	50	3⅜	46	32	1⅞
38	48¾	2⅜	45¼	32	1⅝	52¼	3 7/16	48	32	1⅞
40	50¾	2½	47¼	36	1⅝	54½	3 9/16	50¼	36	1⅞
42	53	2⅝	49½	36	1⅝	57	3 11/16	52¾	36	1⅞
44	55¼	2⅝	51¾	40	1⅝	59¼	3¾	55	36	2
46	57¼	2 11/16	53¾	40	1⅝	61½	3⅞	57¼	40	2
48	59½	2¾	56	44	1⅝	65	4	60¾	40	2
50	61¾	2¾	58¼	44	1¾					
52	64	2⅞	60½	44	1¾					
54	66¼	3	62¾	44	1¾					
56	68¾	3	65	48	1¾					
58	71	3⅛	67¼	48	1¾					
60	73	3⅛	69¼	52	1¾					
62	75¼	3¼	71¾	52	1⅞					
64	78	3⅜	74	52	1⅞					
66	80	3⅜	76	52	1⅞					
68	82¼	3⅜	78¼	56	1⅞					
70	84½	3½	80½	56	1⅞					
72	86½	3½	82½	60	1⅞					
74	88½	3⅝	84½	60	1⅞					
76	90¾	3⅝	86½	60	1⅞					
78	93	3¾	88¾	60	2					
80	95¼	3¾	91	60	2					
82	97½	3⅞	93¼	60	2					
84	99¾	3⅞	95½	64	2					
86	102	4	97¾	64	2					
88	104¼	4	100	68	2					
90	106½	4⅛	102¼	68	2¼					
92	108¾	4⅛	104½	68	2¼					
94	111	4¼	106¾	68	2¼					
96	113¼	4¼	108½	68	2¼					
98	115½	4⅜	110¾	68	2¼					
100	117¾	4½	113	68	2¼					

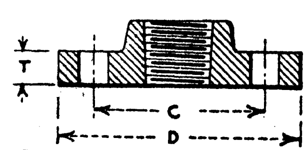


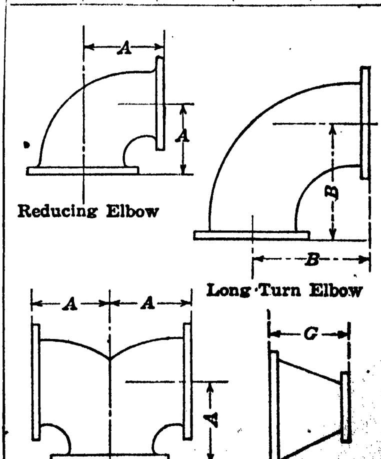
FIG. 174.—Standard flange.

* Flanges for light-weight pipe (50-lb. pressure) have not been standardized (see *E. N. R.*, June 19, 1919, p. 1, 197).

† For bolt and nut weights, see p. 344.

Table 101. Standard Flanged Fittings (Except Short Body Reducers)
(All dimensions in inches)

Pipe size	B	G	Standard weight. Working pressure, 125 lb. per sq. in. A	C*	D*	E*	Min. metal thickness	Extra heavy. Working pressure, 250 lb. per sq. in. A	C*	D*	E*	Min. metal thickness
1	5	.....	3½	1¾	5¾	1¾	7/16	4	2	6½	2	½
1¼	5½	.....	3¾	2	6¼	1¾	7/16	4¼	2½	7¼	2¼	½
1½	6	.....	4	2¼	7	2	7/16	4½	2¾	8½	2½	½
2	6½	.....	4½	2½	8	2½	7/16	5	3	9	2½	½
2½	7	.....	5	3	9½	2½	7/16	5½	3½	10½	2½	9/16
3	7¾	6	5½	3	10	3	7/16	6	3½	11	3	9/16
3½	8½	6½	6	3½	11½	3	7/16	6½	4	12½	3	9/16
4	9	7	6½	4	12	3	½	7	4½	13½	3	⅝
4½	9½	7½	7	4	12½	3	½	7½	4½	14½	3½	⅝
5	10¼	8	7½	4½	13½	3½	½	8	5	15	3½	11/16
6	11½	9	8	5	14½	3½	9/16	8½	5½	17½	4	¾
7	12¾	10	8½	5½	16½	4	⅝	9	6	19	4½	13/16
8	14	11	9	5½	17½	4½	⅝	10	6	20½	5	13/16
9	15¼	11½	10	6	19½	4½	11/16	10½	6½	22½	5	⅞
10	16½	12	11	6½	20½	5	¾	11½	7	24	5½	15/16
12	19	14	12	7½	24½	5½	13/16	13	8	27½	6	1
14	21½	16	14	7½	27	6	⅞	15	8½	31	6½	1⅛
15	22¾	17	14½	8	28½	6	⅞	15½	9	33	6½	1 3/16
16	24	18	15	8	30	6½	1	16½	9½	34½	7½	1¼
18	26½	19	16½	8½	32	7	1 1/16	18	10	37½	8	1⅜
20	29	20	18	9½	35	8	1⅛	19½	10½	40½	8½	1½
22	31½	22	20	10	37½	8½	1 3/16	20½	11	43½	9½	1 9/16
24	34	24	22	11	40½	9	1¼	22½	12	47½	10	1⅝
26	36½	26	23	13	44	9	1 5/16	24	13	....	....	1 13/16
28	39	28	24	14	46½	9½	1⅜	26	14	....	....	1⅞
30	41½	30	25	15	49	10	1 7/16	27½	15	....	....	2
32	44	32	26	16	.....	.....	1½	29	16	....	....	2⅛
34	46½	34	27	17	.....	.....	1 9/16	30½	17	....	....	2¼
36	49	36	28	18	.....	.....	1⅝	32½	18	....	....	2⅜
38	51½	38	29	19	.....	.....	1 11/16	34	19	....	....	2 7/16
40	54	40	30	20	.....	.....	1¾	35½	20	....	....	2 9/16
42	56½	42	31	21	.....	.....	1 13/16	37	21	....	....	2 11/16
44	59	44	32	22	.....	.....	1⅞	39	22	....	....	2 13/16
46	61½	46	33	23	.....	.....	1 15/16	40½	23	....	....	2⅞
48	64	48	34	24	.....	.....	2	42	24	....	....	3
50	66½	50	35	25	.....	.....	2 1/16					
52	69	52	37	26	.....	.....	2⅛					
54	71½	54	39	27	.....	.....	2 3/16					
56	74	56	41	28	.....	.....	2¼					
58	76½	58	42	29	.....	.....	2 5/16					
60	79	60	44	30	.....	.....	2 7/16					
62	81½	62	45	31	.....	.....	2½					
64	84	64	47	32	.....	.....	2 9/16					
66	86½	66	48	33	.....	.....	2⅝					
68	89	68	50	34	.....	.....	2 11/16					
70	91½	70	51	35	.....	.....	2¾					
72	94	72	53	36	.....	.....	2 13/16					
74	96½	74	54	37	.....	.....	2⅞					
76	99	76	56	38	.....	.....	2 15/16					
78	101½	78	58	39	.....	.....	3					
80	104	80	59	40	.....	.....	3 1/16					
82	106½	82	60	41	.....	.....	3⅛					
84	109	84	62	42	.....	.....	3 3/16					
86	111½	86	63	43	.....	.....	3¼					
88	114	88	65	44	.....	.....	3 5/16					
90	116½	90	67	45	.....	.....	3⅜					
92	119	92	68	46	.....	.....	3½					
94	121½	94	69	47	.....	.....	3 9/16					
96	124	96	71	48	.....	.....	3⅝					
98	126½	98	73	49	.....	.....	3 11/16					
100	129	100	74	50	.....	.....	3¾					



Reducing Elbow

Long Turn Elbow

Double Branch Elbow

Reducer

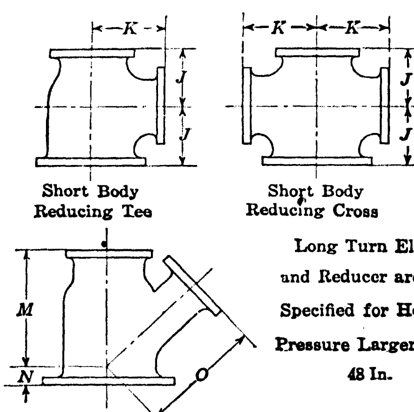
* For elbow, 45° elbow, Tee, Cross, Lateral, see Fig. 154, p. 348.

Long-turn elbow and reducer are not specified for heavy pressure larger than 48 in.

Table 102. Standard Flanged Short Body Reducers*

(All dimensions in inches)

Pipe size	Standard weight. Working pressure, 125 lb.. per sq. in.							Extra heavy. Working pressure, 250 lb. per sq. in.				
	Tees and crosses			Laterals				Tees and crosses†		Laterals†		
	Max. size of outlets	J	K	Max. size of branches	M	N	O	J	K	M	N	O
18	12	13	15½	9	25	1	27½	14	17	31	3	32½
20	14	14	17	10	27	1	29½	15½	18½	34	3	36
22	15	14	18	10	28½	½	31½	16½	20	37	3	39
24	16	15	19	12	31½	½	34½	17	21½	41	3	43
26	18	16	20	12	35	0	38	19	23	....	....	....
28	18	16	21	14	27	0	40	19	24	....	....	....
30	20	18	23	15	39	0	42	20½	25½	....	....	....
32	20	18	24	....	....	....	....	20½	26½	....	....	....
34	22	19	25	....	....	....	....	22	28	....	....	....
36	24	20	26	....	....	....	....	23½	29½	....	....	....
38	24	20	28	....	....	....	....	23½	30½	....	....	....
40	26	22	29	....	....	....	....	25	31½	....	....	....
42	28	23	30	....	....	....	....	26½	33½	....	....	....
44	28	23	31	....	....	....	....	26½	34½	....	....	....
46	30	24	33	....	....	....	....	27½	35½	....	....	....
48	32	26	34	....	....	....	....	29	37½	....	....	....
50	32	26	35									
52	34	27	36									
54	36	29	37									
56	36	29	39									
58	38	31	40									
60	40	33	41									
62	40	33	42									
64	42	34	44									
66	44	35	45									
68	44	35	46									
70	46	37	47									
72	48	40	48									
74	48	40	49									
76	50	42	50									
78	52	43	52									
80	52	43	53									
82	54	44	54									
84	56	47	56									
86	56	47	57									
88	58	48	58									
90	60	50	61									
92	60	50	62									
94	62	52	63									
96	64	53	64									
98	64	53	65									
100	66	55	67									



Short Body Reducing Tee

Short Body Reducing Cross

Short Body Reducing Lateral

Long Turn Elbow and Reducer are not Specified for Heavy Pressure Larger than 48 In.

* For minimum metal thickness allowed, see Table 101.
† Maximum size outlet and branches, same as for Standard Weights.

Gaskets.* Canvass-inserted black-rubber or paper gaskets† are ordinarily used on low-pressure flanged joints. For higher pressure, not exceeding 160 lbs. per sq. in. wire-insertion rubber gaskets $\frac{1}{16}$ in. thick should be used. For pressures exceeding 160 lbs. per sq. in. use corrugated steel gaskets covering the entire ring area inside the bolt holes. To retain the gaskets the flange faces have to be machined in some way. For pressures less than 125 lbs. per sq. in. the plain straight face is commonly employed. Best results are obtained by

* Mark's "Mechanical Engineers' Handbook," p. 878 (McGraw-Hill Book Co., Inc., 1924).
† See page 807.

using a fairly thick gasket on which the bolts will exert sufficient pressure to make the joint tight before the outside edges will meet. Plain face corrugated flanges having coarse concentric grooves, cut with a round nose tool, are used for high pressure joints requiring an exceptionally thick gasket. The corrugations prevent the gasket from blowing out.

SPECIAL PIPE

Expansion joint pipe[30] is used where exposed to excessive expansion and contraction and for carrying gas or water under high pressure. Molded rubber gaskets are slipped over the spigot ends, which are faced and the whole drawn into position by the ring clamp as shown. It is the tightest joint with

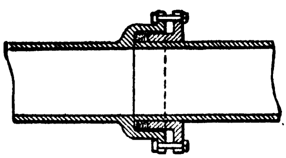


FIG. 175.—Expansion joint pipe.

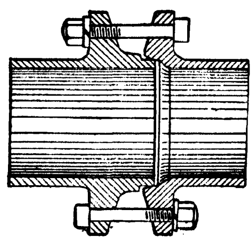


FIG. 176.—Universal joint pipe.

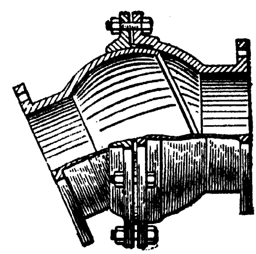


FIG. 177.—Flexible ball joint.

cast-iron pipe, for either gas or water, and will hold the highest pressure the pipe will stand, the makers claim.

Table 103. Weight per Ft., Expansion Joint Pipe (12-ft. Lengths)

Size, in.	4	6	8	10	12	14	16	18	20	24
Standard: weight per ft., lbs.	22	34	47	64	82	125	133	160	190	260
Medium: weight per ft., lbs.	24	38	55	73	95	140	150	180	225	300
Extra heavy: weight per ft., lbs.	26	40	60	80	110	145	175	205	250	350

Threaded pipe can be furnished of A. W. W. A. thickness up to 16 in. diam. Below 6-in. diam. the couplings are standard and correspond to the next larger size of wrought-iron pipe fitting, in 6- to 16-in. sizes, threads and fittings are special.[30] (For fittings, see Marks' "Mechanical Engineers' Handbook," p. 883 (McGraw-Hill Book Company, Inc., 1924), or "Machinery's Handbook," 1924, p. 1504.)

Table 104. New York City Cast-iron High-pressure Fire Pipe*

Diam., in.	Thickness, in.	Unit tensile stress under 300 lbs. pressure per sq. in.	Factor of safety
24	$1\frac{7}{8}$	1920	10.4
20	$1\frac{1}{2}$	2000	10.0
16	$1\frac{1}{4}$	1920	10.4
12	1	1800	11.1
8	$\frac{7}{8}$	1371	14.6

* See also p. 402.

Flexible Ball Joints (American Spiral Pipe Works) (Fig. 177). **Parts are accurately machined, thus making tight joint. Outer bell is reinforced near flange. Retaining flange or ring is constructed to allow a deflection of 20°. It is made to bolt to flange or outer bell, and supports not only body, but entire joint. Flexible joints are discussed on p. 416 also.**

High-pressure Fire Pipe.* Tests made in New York on an ordinary poured lead joint in a 12-in. cast-iron pipe showed that it held up to 750 lb. per sq. in., the highest test pressure obtainable. The three-way and four-way special castings are generally of cast steel; a large safety factor being pro-

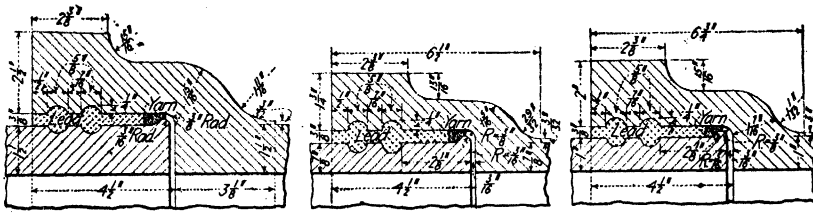


FIG. 178.—Joint for 20-in. high-pressure mains, Manhattan.

FIG. 179.—Joints for 16-in. high-pressure mains, Coney Island.

(Coney Island pipes were tested under 225 lbs.; normal pressure 125 lbs. Size 16 in. and 12 in. Joint dimensions for 16-in. pipe, ¾ and 1 in. thick as shown. Extra thickness provides for salt water corrosion.[31])

vided.[33] Extensive tests on joints used at San Francisco are given in *E. N.* Feb. 18, 1915, p. 290. Philadelphia uses Universal pipe.

Victaulic joint,[34] as developed in England, consists of a hollow steel ring, or housing, slipping over the abutting spigot ends. Within this is an india-rubber ring, U-shaped, so that water pressure within it forces it into water-tight contact with the pipe and the steel housing. A 5° deflection per joint is possible.

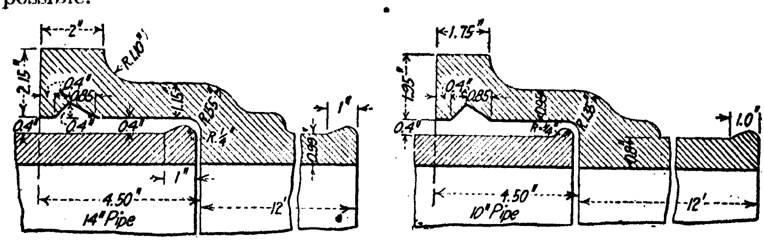


FIG. 180.—Joints for high-pressure mains, Oakland, Cal.
(Pressure, 200 lb. per square inch.[32])

Factory-made joints† consist of hemp and lead inserted at the factory around a mandrel slightly larger than the spigot, so that the spigot will easily enter the bell. This packing is protected during shipping by a concrete mandrel. **The advantages of eliminating lead pouring are obvious.‡ Calking only is needed to produce a tight but resilient joint. Sizes, 1¼ to 6 in.; the 4- and 6-in. sizes in 12-ft. lengths; the others in 5-ft.**

* See also p. 403.
† McWane Cast Iron Pipe Co., Birmingham, Ala.
‡ The advantage is most pronounced in wet trenches where steam explosions may result from use of molten lead.

de Lavaud centrifugal process[36] requires a plant consisting of a cupola, a revolving water-cooled molding machine, an annealing furnace, and a dipping vat. Inside of mold is same diam. as outside of pipe. No bead on spigot. Mold is revolved at high velocity by an impulse water wheel. From the mold, the casting passes to the annealing furnace. These metallurgical processes produce, according to tests, a metal of better quality than other processes. Great possibilities for usefulness and economy, in less material, less labor, and less foundry space, are offered. It sells (1924) about 5 per cent. cheaper per ft. than sand-cast pipe.[38] Indications are that rejections will be low. Smoother interior means greater capacity. This pipe is covered by patents in all countries, the American rights being held by U. S. Cast Iron Pipe and Foundry Co., and National Cast Iron Pipe Co. Superintendent Blomquist of Cedar Rapids after a trial in 1923 reported the pipe as yet too brittle for successful use;[37] recent improvements in composition and processes have yielded better results; in 1926 there are 800 users of this pipe (Ruggles).

Sand-spun pipe[39] is the product of a new centrifugal process of cast-iron pipe manufacture developed through the joint experimental work of a group including R. D. Wood & Co., Philadelphia; American Cast Iron Pipe & Foundry Co., Birmingham, Ala.; Warren Foundry & Pipe Co., New York; Donaldson Iron Co., Emaus, Pa.; Lynchburg Foundry Co., Lynchburg, Va.; and the Glamorgan Pipe & Foundry Co., Lynchburg, Va.

Pipe made by the new process in centrifugal sand-lined molds, according to the manufacturers, has been thoroughly tested in hydraulic presses to 2400 lb. per sq. in. without showing defects. A $\frac{1}{16}$-in. cement-mortar lining may be spun.

French pipe, manufactured by Pont-à-Mousson, conforms to standard specifications except in laying length and phosphorus content. It has been used in Detroit, Holyoke, Kingston, Norwalk and Morristown.

Universal cast-iron pipe* has an iron-to-iron joint consisting of a shallow conical hub with an accurately machined bearing surface and a corresponding short conical-machined spigot, the taper of the spigot being $\frac{1}{2}$ deg. sharper than that of the hub (see Fig. 176). Each hub and each spigot has a pair of bolt lugs, for ordinary pressures, and two pairs equally spaced for high pressures; in the former case, the bolts are at the ends of the horizontal diameter, as laid, and are useful principally for drawing the pipes together when the line is being constructed. Universal pipe was first put on the market in 1900; redesigned in 1908 to increase weight and strength of lugs. A good quality of iron is demanded by the machined joint.† Pipes are subjected to hydrostatic test before coating. Pipe is given the usual pipe dip, except the machined surfaces, which are slushed. Bolts may be preserved by galvanizing, sherardizing, or other coating. In making the joints a paste of white lead is used as a lubricant. No packing, gasket, lead, nor calking is required, and joints are quickly made. Pipes can readily be put together in shallow water, and no "bell holes" or other enlargements of the trench are needed. This joint is slightly flexible and remains tight under moderate

* Central Foundry Co., New York City.
† For the same pressures, pipes are thinner than specified for *A. W. W. A.* standards; the makers claim this possible because of the higher quality of metal.

distortion or slight withdrawal of the spigot; consequently the pipe line can endure settlement and expansion without special devices. Has been used successfully for subaqueous lines, for exposed lines on bridges subject to vibration, and for high-pressure fire service. Pipe lines can be easily taken apart and relaid. The iron-to-iron joint eliminates or at least greatly reduces electrolysis. Straight lengths may be laid to a curve of 150-ft. radius, and 14-in. pipe has been laid on 88-ft. radius. At Holt, Ala., 72 ft. of 12-in. pipe were left unsupported by a washout and sagged 46 in. but remained quite tight under pumping service until the embankment was rebuilt. In Erie, Pa., similarly, a 90-ft. stretch of 12-in. pipe left unsupported by undermining in a flood sagged 4 ft., but remained tight. In Philadelphia high-pressure fire system all joints were tested to 400 lb. while exposed in the trench and required to be absolutely tight; 31 mi. were laid 1908 to 1912. Specifications may be drawn for pipe lines with no allowance for joint leakage. Even with all bolts removed after laying, pipe lines have remained tight under 150 lb. pressure. Total cost of a pipe line, including trenching, does not differ greatly from that of a corresponding line of the usual bell-and-spigot cast-iron pipe with lead joints. The usual varieties of special castings are obtainable; they are class No. 250. Many manufacturers furnish valves and hydrants with the Universal-joint connection. Approval of National Board of Fire Underwriters was given, Oct. 16, 1916. Trouble from thin spots has been reported by superintendents; these result from casting the pipe horizontally. A pipe designed for 125-lb. pressure would not take 100 lb.[40] Bolts have rusted in cinder fill and necessitated substitution of bell-and-spigot pipe.

Table 105. Universal Cast-iron Pipe, Dimensions and Weights
Central Foundry Co., New York City.

Nominal inside diameter	Class No. 100, 100 lb. pressure			Class No. 130, 130 lb. pressure			Class No. 175, 175 lb. pressure			Class No. 250, 250 lb. pressure			Bolt sizes, in.
	Approx. thickness, in.	Estimated weight, lb. per Ft.	Estimated weight, lb. per 6-ft. length	Approx. thickness, in.	Estimated weight, lb. per Ft.	Estimated weight, lb. per 6-ft. length	Approx. thickness, in.	Estimated weight, lb. per Ft.	Estimated weight, lb. per 6-ft. length	Approx. thickness, in.	Estimated weight, lb. per Ft.	Estimated weight, lb. per 6-ft. length	
2	......	....	.....	....	....	.....	0.35	8½	51	0.39	9½	57	½×3½
3	......	....	.....	....	....	.....	0.37	13	78	0.42	14½	87	½×4
4	0.37	18	108	0.40	18¾	112½	0.43	20¼	121½	0.45	21¼	127½	⅝×5
6	0.43	30	180	0.45	31	186	0.47	32	192	0.51	35½	213	¾×6
8	0.47	44¼	265½	0.49	46	276	0.525	49¼	295½	0.58	53¼	319½	⅞×6½
10	0.50	60½	363	0.53	63½	381	0.58	67¾	406½	0.64	74	444	1 ×7½
12	0.53	75½	453	0.57	80½	483	0.62	87	522	0.70	97½	585	1 ×8
14	0.565	94½	567	0.60	99½	597	0.66	107½	645	0.76	124	744	1⅛×9
16	0.60	115½	693	0.65	123	738	0.72	134	804	0.83	156	936	1¼×9½

Lengths lay full 6 ft. All pipe tested with hydrostatic pressure of 300 lb. per sq. in. Gas pipes also tested with compressed air and soap suds. Pipe will be tarred unless ordered.

Simplex prepared joint 5-ft. pipe, 5 ft. long, has bolted male and female joints, in which watertightness and flexibility are secured by lead and jute gaskets. Made by American Cast Iron Pipe Co., 2-, 3-, 4-, and 6-in. diam.; for working pressures up to 150 lb. per sq. in.

Bibliography, Chapter 18. Cast-iron Pipe

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CHAPTER 19

DISTRIBUTION SYSTEMS*

General Requirements. Some important requisites of a satisfactory distribution pipe system are well-laid mains of durable† material, of sufficient capacity to meet all demands with no more than a reasonable drop in pressure; two or more main feeders well located with respect to the system as a whole; all mains interconnected and so controlled by valves that in event of break or for other reason a small section may be put out of commission quickly, with minimum inconvenience to consumers; master meters registering the quantities delivered into the several districts; protection against electrolysis; ample but not too high pressures; freedom from causes of interruption of flow; a good map showing the position, size, material, depth of cover, and date of laying of each main, together with size, location, and kind of each valve, hydrant,‡ air valve, blow-off, and other appurtenance. For sample map, see *E. C.*, Sept. 13, 1922, p. 259.

Requirements of Board of Fire Underwriters. Supply mains shall be so laid as not to endanger one another, be protected against failure at railroad and stream crossings and at other exposed points, be cross-connected and gated about once a mile, and be equipped with air valves at high points and blow-offs at low.

Arterial System. Arteries and secondary feeders shall extend throughout the system. Feeders shall be of capacity required for fire flow, spaced about 3000 ft. and looped. Arterial mains shall not be laid across filled ground and shall be so gated that not more than ¼ mi. within the distribution system shall be affected by a break. All mains shall have sufficient cover to prevent freezing, with a minimum of 2 ft. to prevent injury from traffic.

Pipe sizes of secondary feeders and all cross-connections are fixed by fire requirements, while sizes of supply mains are fixed by ordinary demands. One effective fire stream is equivalent to the maximum domestic requirements of 5000 people (see also p. 571).

Size Limits. Except in small towns, no main used for permanent fire service should be less than 6 in.; in many places 8 in. is the established minimum. In small towns, on lines where no hydrants are required, 2- to 1½-in. cement-lined threaded pipe has proved durable (20 yrs'. use).[2] Damage from a break increases with size of pipe; usual practice limits street mains to 48 in.; there are 66-in. pipes in Brooklyn and 72-in. in Jersey City, about the largest sizes possible without undue interference with other subsurface structures,

* For a valuable general discussion, see "Pipe Distribution Systems," by N. S. Hill, Jr., *J. A. W. W. A.*, Vol. 2, 1915, p. 107. American and British practices are compared in *Water & Water Eng.*, Vol. 25, 1923, p. 443.

† For life and durability of pipe materials, see index.

‡ For spacing see pp. 401 and 457.

particularly in an old street. Only under unusual conditions should there be more than one large main in a street; otherwise a break (*a*) endangers the other pipe, and (*b*) is difficult to locate.

Materials. Cast-iron, steel, cement-lined, wrought-iron, and wooden pipe are used (for the last, see p. 367).

Cast-iron vs. Steel. Cast iron is almost universally used for the smaller sizes, although some western cities, where freight rates increase the cost of heavy pipe, have successfully used steel. In Pasadena, riveted-steel distribution mains from 3- to 20-in. diam. are in use. The Board of Water Supply, New York, laid cast-iron pipes up to 48 in. in city streets, and 66-in. steel pipes. Very large cast-iron pipes are subject to shrinkage and other stresses of unknown amount which render them unreliable under heavy pressures. They should rarely be used above 48 in., where pressures are high, although made up to 84 in. (for breaks, see p. 385). Substitution of several small for one large pipe meets this difficulty, but at considerable increase in cost. For large mains steel should have serious consideration, particularly where pressures are high. When a cast-iron pipe fails, commonly a large piece blows out so that the whole conduit drains rapidly. Steel pipes generally fail by corrosion, causing many small openings; rarely, unless caused by improper emptying or filling (see p. 312), are breaks large enough to cause extensive damage. Electrolysis does greater damage to steel pipes.

Gridiron System. The function of a distribution system is to deliver water in required quantities, and at required pressures, wherever wanted. Test of adequacy is a large fire. If water can be delivered to a hydrant from two directions instead of one, pressure loss is decreased 75 per cent. Cross-connecting mains at some points of intersection and providing feeders $\frac{1}{4}$ to $\frac{1}{2}$ mi. apart with cross-connections every block create a gridiron system which serves the dual purpose of raising hydrant pressures, and, if properly valved, of causing least inconvenience for repairs. The maximum hourly demand should be used as basis for pipe design. Where the waste is a minimum, the demand will be concentrated in 16 hr. (For methods of computing flows in a gridiron system, see "Public Water Supplies," by Turneaure and Russell, 3rd ed., p. 729; also Pardoe in *E. N. R.*, Sept. 25, 1924, p. 517.) Installation of additional paralleling feeders increases pressures greatly. Paralleling a 6-in. line with a 12-in., with cross-connections at 300-ft. intervals, and connections to 8-in. lines in parallel streets, raised some hydrant pressures for 10 streams from 10 to 83 lb.[3]

Advantages of Ample Pipe Sizes and Correct Location. (*a*) Better fire protection. (*b*) If system meets requirements of Fire Underwriters, lower fire-insurance rates. (*c*) Small fluctuations of pressure with varying demands of consumers. (*d*) Long life before replacement, or supplementing of outgrown pipes. Against these advantages must be weighed higher first cost and interest. See "Effect of Distribution System Design upon Fire Insurance Rates," by Stocklmeir, *J. A. W. W. A.*, Vol. 11, 1924, p. 572; and "Standard Schedule for Grading Cities and Towns," prepared by National Board of Fire Underwriters.

Pressures in Distribution Mains. With properly proportioned distribution systems and comprehensive schemes of fire fighting with fire engines,

hydrant pressures of 30 to 40 lb.* are as good as 60 to 80; additional pressure will not be of material advantage to engines; an engine can pump an adequate supply so long as the pressure on the hydrant is sufficient to deliver water to the engine. It is more necessary, from fire insurance standpoint, to have adequate main capacity than high pressure. Higher pressure allows smaller mains, but in such ratio that it is usually more economical to use larger mains, unless pipes are costly and high-gravity pressure easily available. Maintaining high pressures in distribution systems for sprinkler service is questionable; New York Fire Insurance Exchange has refused to consider the city supply as a satisfactory source for sprinkler service, as it is generally metered, and fluctuations are liable, beyond control of the building owner; two independent sources for sprinkler service are demanded before reducing fire rates. In villages and small towns reasonably high pressures in mains will make fire engines unnecessary.

The pressure on the street mains of a city which is necessary to give a satisfactory domestic service depends on heights of buildings. With house pipes of reasonable size, static pressure of 20 lb. on top floor of six-story building gives good flow; this calls for about 50 lb. at curb. It does not pay to try to serve skyscrapers directly from street mains, because of leakages resulting from heavy pressures.† Very tall buildings are divided into a convenient number of tiers of several stories each, and each tier provided with pumping apparatus, tanks, etc., by the owner.

Fire Service Pressures. Common practice in this country in towns having pumped supplies is to raise pressure when signal for fire is received. Henderson compiled data from 143 cities in the United States in *J. A. W. W. A.*, Vol. 9, 1922, p. 584, wherein increase varies from 4 to 55 lb., averaging 24 (see also table in *Public Works*, Vol. 55, 1924, p. 159). High pressures (over 100 lb.) increase waste and put heavy strains on the systems, particularly services and house fixtures. A 12-in. main has been known to burst. In the excitement, engines have been accidentally stopped while changing steam valves for higher pressures. The advent of the automotive pumper, with which most towns are now supplied, eliminates need for raising pressure. Few cities in the United States, carrying fire and domestic supply in same pipes, have hydrant pressure of 100 lb. Average New England pressure is about 75 lb. at hydrant. This will not supply hose over 300 ft. long. Fire pressures as classified by E. G. Hopson, former chief engineer of National Board of Fire Underwriters, Committee of 20: *Low-pressure*, below 45 lb., sufficient for fire-engine or pump supply but inadequate for other service except to very limited extent or in very low buildings. *Medium-pressure* (45 to 70 lb.) sufficient for moderate streams for inside work for buildings up to three or four stories with moderate lengths of hose; also for sprinkler supply in buildings of small to medium height. *High-pressure* (70 to 100 lb.), fairly effective stream through hose lengths up to 300 ft., giving excellent auxiliary service to fire department; fairly adequate for sprinklers in all but buildings of excessive height (1905).

* At hydrant under full flow conditions. Allow 3 lb. loss through hydrant.

† Metcalf[102] estimated that consumption at Akron would increase 35 per cent., if 60 lb. pressure were increased to 100 lb.

Tests of 347 steam fire engines, 1909–1910, by National Board of Fire Underwriters, in 45 cities throughout the country, showed average delivery to be 88 per cent. of rated capacity, based on 100 lb. net water pressure. At serious fires, long lines of hose, and consequently higher pressures, are required; under such conditions, not more than 50 per cent. of rated capacity can be relied on for any length of time.

FIRE PROTECTION*

Fire Streams. The standard fire stream is considered 250 g.p.m., the discharge of a 1⅛-in. smooth nozzle. Hydrant requirements are generally based thereon. In outlying districts of small towns, streams of 150 to 175 g.p.m. are considered reasonable fire protection. Johnson[1a] argues that any fire getting such headway that it cannot be conquered by two streams will leave little after being extinguished by six streams; the value of the streams lies chiefly in saving adjacent properties; for this purpose even small streams are of great value. (Prevention of a small fire becoming, under favoring conditions, a serious conflagration is very important.)

Table 106. Required Fire Flow and Hydrant Spacing

For the Usual Conditions

Population	G.p.m.	Area per hydrant, 1000 sq. ft. (Engines)	Area per hydrant, 1000 sq. ft. (No engines)	Population	G.p.m.	Area per hydrant, 1000 sq. ft., (Engines)
1,000	1,000	120	100	28,000	5,000*	85
2,000	1,500	....	90	40,000	6,000	80
4,000	2,000	110	85	60,000	7,000	70
6,000	2,500	....	78	80,000	8,000	60
10,000	3,000	100	70	100,000	9,000	55
13,000	3,500	....	....	125,000	10,000	48
17,000	4,000	90	55	150,000	11,000	43
22,000	4,500	....	....	200,000	12,000	40

Over 200,000 population, 12,000 g. p. m., with 2000 to 8000 g. p. m. additional for a second fire. The columns headed "Area" show the number of square feet to be served by one hydrant; *i.e.* if the high-value district of a town of 10,000 population contain 800,000 sq. ft., there should be $\frac{800,000}{100,000} = 8$ hydrants in the area, if the Fire Dept. has engines; and $\frac{800,000}{70,000} = 12$ hydrants, if there are no engines.

* Area per hydrant for flow of 5000 g.p.m. and greater without engines, should be 40,000 sq. ft.

Fire Requirements of the National Board of Fire Underwriters are given in Table 106. A municipality unable to meet requirements suffers in insurance rating. Many small communities, below 3000, fail to meet requirements. National Board requires at least 500 g. p. m. in residential districts but 30 per cent. developed, with low buildings.

Hydrant spacing (see also Chap. 20) is specified by National Board of Fire Underwriters as shown in Table 106. Where these requirements cannot be economically met, as in a small town, a hydrant that will furnish 300 g.p.m. under suitable head to any building will afford good fire protection.[1a]

* See also "Fire Prevention and Protection," by A. C. Hutson (1916); Specifications of Inspection Dept., Associated Factory Mutual Fire Insurance Companies; and "Requirements for Small Towns," by Goldsmith, *J. A. W. W. A.*, Vol. 12, 1924, p. 168.

Table 107 shows importance of having hydrants near buildings to be protected. In a small system, eight two-way hydrants per mi., may be estimated.[1b]

Table 107. Height and Volume of a 1⅛-in. Stream Flowing from a Smooth Nozzle (E. V. French)[1c]

Length of hose, ft.	Limit of height, ft.	Discharge, g.p.m.
100	67	250
200	59	222
300	52	206
400	44	188
500	40	178
700	33	158

High-pressure fire systems* involve heavy pipes (see p. 394) and hydrants, with booster pumps to eliminate fire-department pumpers. The source of supply is generally that used for other purposes. New York discarded the use of sea water after its effects on pumping equipment had become noticeable. High-pressure fire systems to protect high-value districts have been installed in New York,[5] Boston,[6] Philadelphia,[7] Cincinnati,[8] Cleveland,[7] Toledo,[9] Baltimore,[7] Oakland,[10] and San Francisco,[11] Buffalo, Detroit, Miami, Jacksonville, Atlantic City, Toronto, Winnipeg. Operating pressures in several instances are 250 to 300 lbs. per sq. in. For results of operation see *E. C.*, Jan. 10, 1917, p. 36. For leakage tests on New York high-pressure systems, see *J. A. W. W. A.*, Vol. 5, 1918, p. 44.

Fire-protection costs are commonly 25 to 50 per cent. of the total; for small towns they rise to 65 or 75 per cent., while in large cities they may be as low as 20 per cent.; Metcalf, Kuichling, and Hawley[4] recommend allocation on basis: per cent. $= \dfrac{147}{x^{0.31}} - 12.1$, where x = population in thousands.[4] This has an important aspect when apportioning charges of private companies. Capital cost is largely and operating cost but little, affected by fire requirements, which generally amount to 1 per cent. of total consumption.[12] See discussion of equitable hydrant rentals by Alvord, *J. A. W. W. A.*, Vol. 1, 1914, pp. 95 and 538.

Fire-protection Service. See report of Committee, *J. A. W. W. A.*, Vol. 6, 1919, p. 679.

Dependability of system is an important consideration in fire protection. This may involve auxiliary pumping equipment to avoid stoppage caused by breakdown, a pumping station† secure against floods and fires, dual sources of power where electric drive is employed, more than one pipe line from source of supply, and mains laid at non-freezing depths. Interruptions and failures of water supplies constitute a fire hazard and are very frequent. In 1920, the Inspection Department of Associated Factory Mutual Fire Insurance Companies compiled a list of all such failures; breaks in large mains, ice

* See also "High Pressure Fire Systems from the Underwriters' Viewpoint," by G. W. Booth, *J. N. E. W. W. A.*, Vol. 36, 1922, p. 495.

† See p. 469.

in suction line, frozen lines, breaking of reservoir dam, breakdown of pumping equipment, fires, and floods are among the causes.

Industrial Fire Protection. Municipal systems are laid only to property lines. Where an industry comprises several buildings scattered over a large yard, the company must lay piping which conforms to usual waterworks practice for city streets. The Associated Factory Mutual Insurance Companies, and the National Board of Fire Underwriters have formulated rules for yard installations (see their manuals), and an A. W. W. A. Committee reported in 1919 on Private Fire Protection Service (see *J. A. W. W. A.*, Vol. 6, 1919, pp. 679–782).

Connections are often made, for added protection, to second source, *e.g.*, some convenient stream, commonly polluted. Although cross-connection always contains a check valve to prevent water from the mill system flowing into the municipal mains, lack of tightness in the valve has often allowed polluted water to get into the distribution system and has been the cause of typhoid outbreaks. Many cities and states now forbid cross-connections (see also p. 452). Burnham[122] uses a double-check valve, without complaints.

Sprinkler systems are subject to elaborate regulations by National Board of Fire Underwriters and Associated Factory Mutual Insurance Companies. Sprinkler systems are installed by the Grinnell Co., Automatic Sprinkler Corp. of America., New York City, and others. They provide the most effective fire protection known; of 28,560 fires in buildings so protected, 84 per cent. were extinguished with 10 or less sprinkler heads opening. The maximum demand per head is estimated at 300 g.p.m. Only about 5 per cent. of the fires demanded more than 1000 g.p.m. Although no records are available, similar fires in buildings not protected by sprinklers demanded many times this quantity of water. Sprinkler system is dependent upon a reliable and adequate supply,, demanding two sources, such that a pressure of 12 lb. is maintained on highest line of sprinklers while 500 g.p.m. are flowing from nearest hydrant.[44]

LAYING CAST-IRON PIPE*

Bedding Cast-iron Pipes. Cast-iron pipes must be very carefully bedded so as to have uniform support under bottom and must never rest on a boulder, point of ledge rock, or similar relatively unyielding object. Pipe's capacity to bear refill load can be materially increased by thorough tamping of refill at and below horizontal diam.; but too great dependence on careful refilling is liable to lead to trouble in places where excavations will be made close to the pipe. Probably more unavoidable breaks have been caused by unwittingly or carelessly permitting a cast-iron pipe to rest on a rock or other hard unyielding object than by any other cause.† In rock, the trench bottom, after being trimmed, should be covered with a small depth of earth. Each 12-ft. pipe of 16-in. or larger diam. is supported on two short planks, or blocks, one just back of the bell and the other near the spigot end, well bedded on the earth. Each pipe is held in position and brought to bearing on each block by two wooden wedges, of 3 × 3 or 4 × 4 × 12 in. long, according to the

* For steel pipe, see p. 326; for wood pipe, pp. 361 and 365.
† Settlement also has been a prominent cause of breaks.

size of pipe. Blocking and wedges are placed under special castings and valves also.

Care in Laying. Pipes should be clean inside when put in trenches; open ends should be plugged when work is stopped to prevent stones rolling inside, or other objects being put in, and to keep out animals. Backfilling should contain no ashes, cinders, or other corrosive materials. Rocks should not be rolled into trenches and allowed to drop on pipes. Large rocks should not be permitted in the backfill close to the pipe. All plugs at blanked openings and all sharp bends in soft ground should either be strapped or secured by masses of concrete. Sometimes sharp curves are blocked against the side of the trench. For soft ground at New Orleans, batter piles were used (see *E. N. R.*, May 15, 1924, p. 855).

Inspection. Pipe laying should be constantly, conscientiously, and intelligently inspected, and all pipes, specials, valves, and other fittings should be carefully examined for incipient cracks and other defects just before laying.

Depth of laying depends primarily on climatic conditions, secondarily on prevention of molestation, and thirdly on interference with other subsurface structures. In tropical countries cross-country conduits are often laid on the surface. In cities sewer connections and other subsurface obstructions often influence. Depth of cover over top varies from 2.5 ft. in the South to between 5.5 and 7 ft. in Canada, New England,* and other equally cold sections, where cover serves as protection from freezing; it should be greater in loose, gravelly soil than in compact, clayey soil.† In England 3 ft. is considered minimum safe depth.[13] Frost protection on bridges and other exposed location is important (see p. 419). In laying pipe through unimproved streets, allow for relation of pipe to probable grade, to assure sufficient cover.

Trench Dimensions. Usual width is 1 ft. greater than internal diam. of pipe, with 2 ft. as minimum width. As the depth of trench is fairly constant, bids for earthwork are commonly taken on lin. ft. basis. In such cases,

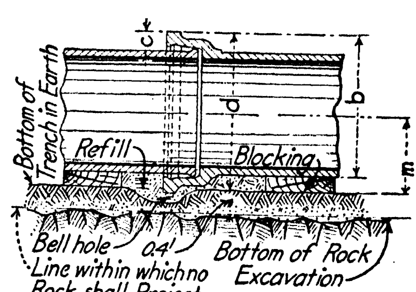


FIG. 181.—Trench dimensions, Table 108.

trench dimensions are of less moment. In Syracuse, N. Y., it is proposed to receive bids on basis of depth of laying, following practice in sewer work, for a cross-country gravity line involving heavier cuts than usual. Table 108 gives the Boston standards. For large pipes, enlargements of trench (commonly called bell holes) are made at each joint to afford room for calking.

* See report of committee, *J. N. E. W. W. A.*, Vol. 27, 1913, p. 160 and table, *J. N. E. W. W. A.* Vol. 23, 1909, p. 435.

† Frost penetrates more deeply under roadways than in uncompacted soil, as in fields and parkways.

To obtain calking space beneath pipes in wet trench, a bottomless box has been used, the sand removed, and pump suction placed in sump so formed.

Table 108. Trench Details, Solid Lead Joints, and Spacing of Air Valves for Cast-iron Pipe Lines

Metropolitan (Boston) Waterworks Standards

Based on New England Water Works Standard Pipe

Pipe diam., inches	Trench dimensions, ft., Fig. 181								Air valves. Use 1 in. up to following lengths in ft.‡	Solid lead joint	
	Width of trench at "A line"‖		d	c*	b†	m		Wood blocking size, in.		Extra lead lb.	Allowance§
	Earth	Rock				Earth	Rock				
4	2.0	3.5	—	0.14	0.55	—	—	....	Any length	....	......
6	2.0	3.5	—	0.15	0.73	—	—	....		....	......
8	2.0	4.0	—	0.16	0.92	—	—	....		....	......
10	2.0	4.0	—	0.16	1.10	—	—	....		....	......
12	2.0	4.0	1.4	0.17	1.28	0.65	0.98	....		9	$0.40
14	2.3	4.0	1.6	0.17	1.47	0.75	1.08	....		11	0.50
16	2.5	4.5	1.8	0.19	1.66	0.90	1.23	2×10×30		20	0.90
20	2.7	4.5	2.2	0.21	2.03	1.10	1.43	2×10×30		23	1.00
24	3.0	5.0	2.5	0.22	2.39	1.25	1.58	2×10×30		28	1.20
30	3.5	5.5	3.2	0.23	2.95	1.60	1.93	3×10×36	5000	45	2.00
36	4.0	6.0	3.7	0.25	3.50	1.85	2.18	3×10×36	3500	54	2.50
42	4.5	6.5	4.3	0.27	4.04	2.13	2.46	3×10×42	2600	78	3.50
48	5.0	7.0	4.8	0.29	4.59	2.40	2.73	3×10×48	2000	88	4.00
54	5.5	7.5	5.4	0.30	5.12	2.70	3.03	3×12×54	1600	115	5.25
60	6.0	8.0	5.9	0.31	5.67	2.95	3.28	3×12×60	1300	130	6.00

* c computed for thinnest pipe. † b computed for thickest pipe. ‡ Use 1½-in. valve for greater length. § Lead at 4½ cts. per lb. ‖ "A line" is bottom of trench in earth; line within which no rock shall project, in rock.

If joint space is unusually narrow, or pipes are deflected extremely, or great rigidity is desired, or for other special reasons, yarn is sometimes omitted and the joint made solid with lead. In the latter case a little clay luting is used inside the pipe to stop the lead from flowing through, or a single small strand of yarn is driven to the bottom of the joint.

Cost data are best expressed in man-hours, although allowances must be made for different rates of work in various localities. Tables for estimating

Table 109. Unit Costs of Laying Cast-iron Pipe
(*a*) Labor at 50 Cents per Hour, (*b*) Cast-iron Pipe at $50 per Ton

Cost per Linear Foot

Size, in.	Lead & hemp costs	Total labor costs	Class A			Class B			Class C			Class D		
			Wt. per ft., tons	Cost per ft.	Total cost	Wt. per ft., tons	Cost per ft.	Total cost	Wt. per ft., tons	Cost per ft.	Total cost	Wt. per ft., tons	Cost per ft.	Total cost
4	$0.05	$0.34	0.010	$0.50	$0.89	0.0109	$0.55	$0.94	0.0117	$0.59	$0.98	0.0125	$0.63	$1.02
6	0.07	0.41	0.0154	0.77	1.25	0.0167	0.83	1.31	0.0179	0.90	1.38	0.0192	0.96	1.44
8	0.09	0.48	0.0215	1.08	1.65	0.0238	1.19	1.76	0.0260	1.30	1.87	0.0279	1.40	1.97
10	0.12	0.55	0.0286	1.43	2.10	0.0319	1.60	2.27	0.0354	1.77	2.44	0.0384	1.92	2.59
12	0.15	0.62	0.0363	1.82	2.59	0.0411	2.05	2.82	0.0459	2.30	3.07	0.0500	2.50	3.27
16	0.21	0.93	0.0542	2.71	3.85	0.0625	3.13	4.27	0.0719	3.60	4.74	0.0792	3.96	5.10
20	0.26	1.28	0.0750	3.75	5.29	0.0875	4.38	5.92	0.104	5.20	6.74	0.115	5.75	7.29
24	0.35	1.87	0.102	5.10	7.32	0.117	5.85	8.07	0.139	6.95	9.17	0.153	7.65	9.87

Rate of work is from Catalog of U. S. Cast Iron Pipe & Foundry Co. Lead at 5 cts. per lb.

costs in terms of man-hours dependent on a stipulated rate of work are given by George Wehrle in *Gas Age*, Dec. 15, 1917, p. 553.

Table 109 covers only ordinary costs. Table 109 is based on units of 50; calculations to any other rate may be readily made. *Example:* What will be ordinary costs to Contractor for 10 in. class C pipe, with labor at 75 cts., cast-iron pipe at $70 per ton, and lead at 6 cts. per lb.? Labor $= 0.55 \times \frac{75}{50} =$ $0.83; cast-iron $= 1.77 \times \frac{70}{50} =$ $2.12; lead $= 0.12 \times \frac{6}{5} =$ $0.15; total = $3.10. Costs should be increased for rock and wet excavation, for caring for underground structures, for removing and replacing paving, for special difficulties, and for other unusual expenses. Allow also for contractors' profits and for costs of engineering, financing, etc.

Water Mains and Sanitary Sewers. Sewers are generally built at the centers of streets; water and gas mains, near the curbs. However, in wide boulevards, there may be a dual system of water mains and sewers, one of each under each sidewalk.* In some cities where water supply and sewer construction are under one department, the two pipes are in the same trench, although public opinion generally favors separate trenches.[123] To save expense, New York City excavates the backfill from sewer trenches and lays water mains and service pipes with a cover of 4 ft. several months after the sewers are laid. The sewer trenches are wide enough to pass manholes without using specials. In Cleveland, a bench 8 in. wide is cut in the side of sewer trench not less than 1.5 ft. above the sewers and 5 ft. horizontally therefrom, for house connections only; separate trenches are required for water mains.[15] Western New York Water Co. found that settlement of sewer had broken water pipes in same trench.[16]

Trenching Machines.† Water Department at Erie, Pa.,[17] completed with machine 2.5 mi. of 6-in. and 1.5 mi. of 12-in. water-main trenches in wooded or frozen ground with shale at bottom, between Feb. 1 and Oct. 5, 1917, at a cost far below recent costs of hand work, even in 1915. Speed on trenches 5.5 and 6 ft. deep, 2 ft. wide, was 3 to 3⅔ ft. per min. This saved more than half its first cost and compensated for scarcity of labor. Pawling & Harnischfeger wheel type, cost $5650 f.o.b., Erie; buckets adjustable to trenches 11.5 to 54 in. wide and 4.5 to 12 ft. deep; driven by four-cylinder, four-cycle, 40-hp. gasoline engine. Ordinarily run by operator and one helper. Backfilling, by team and scraper. Machine in 1620 ft. of gravel worked at 8.2 cts. per ft., for labor, fuel, and sheeting. Through cutover land; full of roots, 682 ft. of trench were dug in 4 hr., at 1.1 ct. per ft., with three men and 15 gal. gasoline. Speed record, 660 ft. in 3 hr. at 0.75 ct. per ft., $3.02 for gas; $1.88 for wages of operator and helper. With 18 in. of frost in ground, 7220 ft. of 2 × 5.5-ft. trench were dug at average speed of 3 ft. per min. After season's use, machine showed little wear. At Garfield,[18] N. J., through clay, hardpan, bowlders, and occasional layers of red shale, machine tore out most of the material without injury to itself. When blasting, the machine's elevating wheel was hoisted to surface, chained logs were put over powder holes, and blast fired without damage and without loss of time in moving. In loamy or clayey soils, 500 to 700 ft. of trench 6 to 8 ft. deep were dug in 9 hr.

* See *E. N. R.*, Aug. 27, 1925, p. 333.
† See "Engineering of Excavation," by G. B. Massey (Wiley, 1924).

In Minneapolis,[19] in 1917, Austin excavator, costing $10,000, dug trenches up to 72 in. wide. Small work by hand. Day labor. Operating cost for 66 working days on trench averaging 11 ft. deep, for 54-in. main, was $3665. Repairs, $808; coal and oil, $549; labor, $2308; 39,200 cu. yd. averaged 9.3 cts. Backfilling by small steam shovel cost, for 48-in. main 9520 ft. long, $1146, or 0.48 ct. per cu. yd. Trench section was 75 sq. ft. gross; total backfill, 23,730 cu. yd. Backfilling plus hauling away 6000 cu. yd. excess material cost 0.212 ct. per cu. yd.

For epitome of opinions pro and con, see results of questionnaire in *E. N. R.*, May 3, 1917, p. 258. See also use of machines in distribution system, M. Mitchell and B. Siems, *J. A. W. W. A.*, Vol. 9, 1922, pp. 1, 172.

Railroad Crossings and Cemeteries. Culverts are often used to carry mains under tracks where a break would be difficult to repair and might cause damage. Steel pipes are sometimes encased in concrete. At Charleston, W. Va., mains are encased in old wrought-iron or steel pipe of larger diam. For method of driving pipe employed at Newark, see *E. C.*, Nov. 16, 1921, p. 461. Where a pipe is carried on an overhead bridge, provisions for expansion and frost protection must be made. A tunnel was driven at a depth of 43 to 57 ft. under a cemetery at Detroit, Mich., to avoid open cut.[20] A tunnel, 8 ft. wide and 20 ft. high was excavated under 13 railroad tracks at Cleveland, and two 48 and one 36-inch pipes, placed in a vertical row.

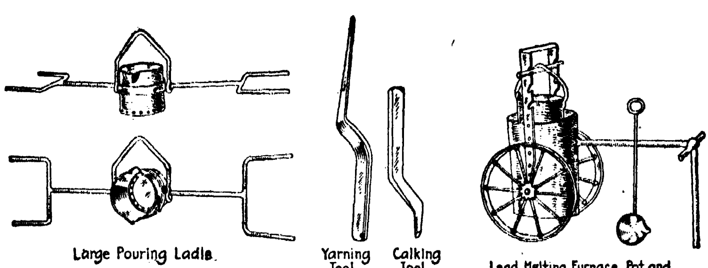


FIG. 181A.—Equipment for lead joints.

Curves. Changes in alinement are effected by deflecting each length of straight pipe so far as the free play in the bell permits for long-radius curves, or by use of specials for short-radius curves (see Table 111). On one section of a 48-in. line in Brooklyn where there were many subsurface obstructions, many joints were opened $1\frac{1}{4}$ in., and an exceptional few $1\frac{3}{4}$ in., to secure deflection. A 24-in. line on the surface through Medicine Canyon, Oklahoma, 30,000 ft. long, with many dips, has no specials.[21] Deflections are recommended where space is available on the score of economy in costs (special bends cost about 75 per cent. more than straight pipe) and reduced friction head. Where both horizontal and vertical deflections occur, economy results from one special. It is not uncommon to find each bend made separately, due to difficulty in computing angle of skewed special.

Table 110. Openings of Cast-iron Bell-and-spigot Pipe Joints in Laying Curved Lines with Straight Pipes 12 Ft. in Length

Class E., N. E. W. W. A. Standard

Joint opening (in.) = sine of angle subtended by 12-ft. chord × outside diam. of pipe (in.)
Deflection of pipe (ft.) = sine of angle subtended by 12-ft. chord × 12 ft.

Pipe diam.			4"	6"	8"	10"	12"	14"	16"	20"	24"	30"	36"	42"	48"	60"	72"	84"	Sine of angle subtended by 12-ft. chord	
Degree of* curve	Radius of curve, ft.	Deflection of pipe in ft.	Horizontal opening of joints of pipe in inches, along axis of pipe																Nat.	Log
	100	1.440	0.59	0.84	1.09	1.35	1.61	1.86	2.11	2.62	3.13	.....	.....	.....	.....	.....	.....	.....	0.120	9.0784
	150	0.960	0.39	0.56	0.73	0.89	1.07	1.24	1.41	1.75	2.09	2.59	3.10	.....	.....	.....	.....	.....	0.0800	8.9024
	200	0.720	0.29	0.42	0.55	0.67	0.80	0.93	1.06	1.30	1.56	1.94	2.32	2.69	3.07	.....	.....	.....	0.0607	8.7829
	250	0.576	0.25	0.35	0.46	0.56	0.67	0.78	0.88	1.09	1.30	1.62	1.94	2.24	2.55	.....	.....	.....	0.0480	8.6811
	300	0.480	0.20	0.28	0.36	0.44	0.53	0.62	0.70	0.87	1.04	1.30	1.55	1.80	2.04	.....	.....	.....	0.0399	8.6010
19	303	0.475	0.19	0.28	0.36	0.44	0.53	0.61	0.70	0.87	1.03	1.28	1.53	1.77	2.02	.....	.....	.....	0.0396	8.5977
18	320	0.451	0.18	0.26	0.34	0.42	0.50	0.58	0.66	0.82	0.97	1.22	1.45	1.68	1.92	.....	.....	.....	0.0375	8.5739
17	338	0.426	0.17	0.25	0.32	0.40	0.48	0.55	0.63	0.77	0.92	1.16	1.37	1.58	1.79	.....	.....	.....	0.0355	8.5501
16	359	0.401	0.16	0.23	0.30	0.37	0.45	0.52	0.59	0.73	0.87	1.08	1.29	1.50	1.70	.....	.....	.....	0.0344	8.5240
15	383	0.376	0.15	0.22	0.28	0.35	0.42	0.48	0.55	0.68	0.82	1.01	1.21	1.40	1.60	.....	.....	.....	0.0313	8.4959
14	410	0.351	0.14	0.21	0.26	0.32	0.39	0.45	0.51	0.63	0.75	0.93	1.12	1.30	1.49	1.79	2.16	2.52	0.0280	8.4464
13	442	0.326	0.13	0.19	0.25	0.30	0.36	0.42	0.47	0.59	0.70	0.87	1.05	1.22	1.39	1.66	2.02	2.34	0.0260	8.4145
12	478	0.301	0.12	0.18	0.23	0.28	0.33	0.39	0.44	0.55	0.65	0.81	0.97	1.13	1.28	1.60	1.94	2.25	0.0251	8.3997
11	522	0.276	0.11	0.16	0.21	0.25	0.31	0.36	0.40	0.50	0.60	0.75	0.89	1.03	1.17	1.47	1.78	2.07	0.0230	8.3615
10	574	0.251	0.10	0.15	0.19	0.23	0.28	0.33	0.36	0.45	0.55	0.68	0.81	0.94	1.06	1.34	1.61	1.88	0.0209	8.3202
9	637	0.226	0.09	0.14	0.17	0.21	0.25	0.30	0.32	0.41	0.50	0.62	0.73	0.86	0.95	1.20	1.45	1.69	0.0188	8.2751
8	717	0.201	0.08	0.12	0.15	0.19	0.22	0.27	0.28	0.37	0.44	0.54	0.65	0.77	0.84	1.07	1.29	1.51	0.0167	8.2236
7	819	0.176	0.07	0.10	0.13	0.16	0.20	0.24	0.25	0.32	0.38	0.47	0.57	0.66	0.74	0.93	1.12	1.32	0.0146	8.1659
6	955	0.151	0.06	0.09	0.11	0.14	0.17	0.20	0.21	0.28	0.32	0.40	0.49	0.56	0.64	0.81	0.97	1.13	0.0126	8.0992
5	1146	0.126	0.05	0.08	0.09	0.12	0.14	0.17	0.19	0.24	0.27	0.34	0.41	0.47	0.54	0.67	0.81	0.95	0.0105	8.0200
4	1433	0.101	0.04	0.06	0.08	0.10	0.11	0.13	0.15	0.18	0.22	0.27	0.32	0.38	0.43	0.54	0.65	0.75	0.00838	7.9231
3	1910	0.075	0.03	0.05	0.06	0.08	0.08	0.10	0.11	0.14	0.17	0.21	0.25	0.29	0.32	0.40	0.49	0.57	0.00628	7.7982
2	2865	0.050	0.02	0.03	0.05	0.05	0.05	0.07	0.08	0.10	0.11	0.15	0.17	0.19	0.21	0.27	0.33	0.38	0.00422	7.6251
1	5730	0.025	0.01	0.01	0.02	0.02	0.03	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.11	0.13	1.16	0.19	0.00209	7.3211
Depth of bells				3"			3.5"			4"		4.5"		5"		5.5"	5.5"	5.5"	....	....
Outside diam., inches			4.78	6.92	9.06	11.20	13.3	15.4	17.5	21.6	25.9	32.2	38.5	44.8	51.1	63.8	77.3	90.1	....	....

For close computations the first three columns relate strictly to the outsides of the pipes on the insides of curves. May be used for both horizontal and vertical curves. Values above heavy broken line may be possible, but are undesirable for good work. (J. H. Lance recommends ¼ in. as least thickness of lead at bell face. Then D = deflection per pipe length in in. = maximum degree of curve, divided by 3.33. The maximum curvature used on a 30-in. pipe = 12° 30'; and for a 12-in. pipe, 18°. By using 6-ft. laying lengths, D = degree of curvature divided by 6.66. (*E. N.*, Sept. 29, 1910, p. 339.))

* Note that this is based on 100-ft. chords. D = twice the angle whose sine is 50/radius.

Table 111. Combinations of Pipe Curves* (Special Castings)

N.E.W.W.A. Standard

Angles, Tangents, Chords, and Arcs, for Pipes from 24 to 60 in. in Diam.

Deflection, angles, ° ′ ″	Diam. in.	Combinations of curves†	Radii, ft.	Tangents, ft.‡		Chords, ft.	Length, arcs, ft.
5 37 30	30 to 60(*a*)	1/64	40	1.97	1.97	3.92	3.93
11 15 00	24 to 60	1 1/32	20	1.97	1.97	3.92	3.93
11 15 00	30 to 60	2 1/64	40	3.94	3.94	7.84	7.85
16 52 30	30 to 60	3 1/64	40	5.93	5.93	11.74	11.78
16 52 30	30 to 60	1/64, 1/32	40, 20	4.61	3.30	1.82	7.85
22 30 00	24 to 30	2 1/32	20	3.98	3.98	7.80	7.85
22 30 00	24 to 30	1/16	10	1.99	1.99	3.90	3.93
22 30 00	30 to 60	4 1/64	40	7.96	7.96	15.61	15.71
22 30 00	36 to 60	2 1/32	20	3.98	3.98	7.80	7.85
22 30 00	36 to 60	1/16	15	2.98	2.98	5.85	5.89
28 7 30	30 to 60	5 1/64	40	10.02	10.02	19.44	19.63
28 7 30	36 to 60	1/64, 1/16	40, 15	5.98	4.01	9.71	9.82
33 45 00	24 to 30	3 1/32	20	6.07	6.07	11.61	11.78
33 45 00	24 to 30	1/32, 1/16	20, 10	4.70	3.38	7.74	7.85
33 45 00	30 to 60	6 1/64	40	12.13	12.13	23.22	23.56
33 45 00	36 to 60	3 1/32	20	6.07	6.07	11.61	17.78
33 45 00	36 to 60	1/32, 1/16	20, 15	5.38	4.73	9.67	9.82
39 22 30	30 to 60	7 1/64	40	14.31	14.31	26.95	27.49
39 22 30	36 to 60	1/64, 1/32, 1/16	40, 20, 15	8.40	5.86	13.45	13.74
45 00 00	24 to 30	4 1/32	20	8.28	8.28	15.31	15.71
45 00 00	24 to 30	2 1/16	10	4.14	4.14	7.65	7.85
45 00 00	24 to 30	1/8	5	2.07	2.07	3.83	3.93
45 00 00	30 to 60	8 1/64	40	16.57	16.57	30.61	31.42
45 00 00	36 to 60	4 1/32	20	8.28	8.28	15.31	15.71
45 00 00	36 to 60	2 1/16	15	6.21	6.21	11.48	11.78
45 00 00	36 to 60	1/8	75	3.11	3.11	5.74	5.89
50 37 30	30	1/64, 2 1/16	40, 10	7.55	4.92	11.33	11.78
50 37 30	36 to 60	1/64, 2 1/16	40, 15	9.44	7.25	15.12	15.71
50 37 30	36 to 60	1/64, 1/8	40, 75	6.60	3.75	9.44	9.82
56 15 00	24 to 30	1/32, 2 1/16	20, 10	7.17	5.58	11.26	11.78
56 15 00	24 to 30	1/32, 1/8	20, 5	5.33	2.91	7.36	7.85
56 15 00	36 to 60	1/32, 2 1/16	20, 15	8.94	8.14	15.10	15.71
56 15 00	36 to 60	1/32, 1/8	20, 7.5	6.29	4.30	9.38	9.82
61 52 30	30	1/64, 1/32, 2 1/16	40, 20, 10	10.58	6.59	14.87	15.71
61 52 30	30	1/64, 1/32, 1/8	40, 20, 5	8.91	3.84	11.24	11.78
61 52 30	36 to 60	1/64, 1/32, 2 1/16	40, 20, 15	12.24	9.34	18.57	19.63
61 52 30	36 to 60	1/64, 1/32, 1/8	40, 20, 7.5	9.74	5.21	13.04	13.74
67 30 00	24 to 30	2 1/32, 2 1/16	20, 10	10.19	7.51	14.79	15.11
67 30 00	24 to 30	3 1/16	10	6.68	6.68	11.11	11.78
67 30 00	24 to 30	1/16, 1/8	10, 5	5.10	3.75	7.40	7.85
67 30 00	36 to 60	2 1/32, 2 1/16	20, 15	11.78	10.44	18.49	19.63
67 30 00	36 to 60	3 1/16	15	10.02	10.02	16.67	17.67
67 30 00	36 to 60	1/16, 1/8	15, 75	1.65	5.63	11.09	11.78
73 7 30	30	1/64, 3 1/16	40, 10	10.31	7.57	14.45	15.71
73 7 30	30	1/64 1/16, 1/8	40, 10, 5	8.78	4.48	10.95	11.78
73 7 30	36 to 60	1/64, 3 1/16	40, 15	13.54	11.25	19.95	21.60
73 7 30	36 to 60	1/64, 1/16, 1/8	40, 15, 7.5	11.24	6.61	14.60	15.71
78 45 00	24 to 30	1/32, 3 1/16	20, 10	10.12	8.40	14.36	15.71
78 45 00	24 to 30	1/32, 1/16, 1/8	20, 10, 5	8.63	5.16	10.88	11.78
78 45 00	36 to 60	1/32, 3 1/16	20, 15	13.27	12.41	19.86	21.60
78 45 00	36 to 60	1/32, 1/16, 1/8	20, 15, 7.5	11.03	7.54	14.52	15.71
84 22 30	30	1/64, 1/32, 3 1/16	40, 20, 10	13.88	9.59	17.63	19.63
84 22 30	30	1/64, 1/32, 1/16, 1/8	40, 20, 10, 5	12.40	6.20	14.40	15.71
84 22 30	36 to 60	1/64, 1/32, 3 1/16	40, 20, 15	16.97	13.91	22.96	25.53
84 22 30	36 to 60	1/64, 1/32, 1/16, 1/8	40, 20, 15, 7.5	14.77	8.22	17.93	19.63
90 00 00	24 to 30	8 1/32	20	20.00	20.00	28.28	31.42
90 00 00	24 to 30	4 1/16	10	10.00	10.00	14.14	15.71
90 00 00	24 to 30	2 1/8	5	5.00	5.00	7.07	7.85
90 00 00	30 to 60	16 1/64	40	40.00	40.00	56.57	62.83
90 00 00	36 to 60	8 1/32	20	20.00	20.00	28.28	31.42
90 00 00	36 to 60	4 1/16	15	15.00	15.00	21.21	23.56
90 00 00	36 to 60	2 1/8	7.5	7.50	7.50	10.61	11.78

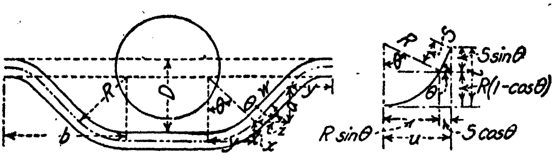
* Bends used in table: 1/64 (angle subtended at center = 5 5/8°); 1/32 (11 1/4°); 1/16 (22 1/2°); 1/8 (45°). Radii, tangents, chords, and arcs are of the center line.

† Whole numbers in this column refer to number of curves required. (*a*) Inclusive in all cases.

‡ Unequal for curves of more than one radius.

The table assumes castings are true to drawings and accurately laid; actually they are likely to vary slightly. Deflections between those tabulated may be made by "opening the joints."

Table 112. Trigonometric Functions of Pipe Curves (Special Castings) for Siphons and Offsets



$a = D - 2x$ $\quad b = 2y + z$

$w = \dfrac{a}{\sin\theta}$ $\quad t = R(1 - \cos\theta) + S\sin\theta$

$z = \dfrac{a}{\tan\theta}$ $\quad u = R\sin\theta + S\cos\theta$

For ¼-curves & 4″ to 12″ ⅛-curves

S In.	S Ft.	S sin θ	S cos θ
4	0.33	0.236	0.236
8	0.67	0.667	0.0
10	0.83	0.833	0.0
12	1.00	1.000	0.0

For S see tables giving dimensions of special castings. (N. E. or Am. W. W. Assn. Tables.)

Curve	Angle θ	Pipe diam., in.	Rad. of curve, R ft.	Natural sine For rad. = 1	Natural sine For rad. of curve	Natural cos For rad. = 1	Natural cos For rad. of curve	R(1−cos θ)	Natural tan For rad. = 1	Natural tan For rad. of curve
¼	90°	4–12	1.33	1.00	1.333	0	0	1.333	∞	∞
		14	1.50	1.00	1.500	0	0	1.500	∞	∞
		16–20	2.00	1.00	2.000	0	0	2.000	∞	∞
		24	2.50	1.00	2.500	0	0	2.500	∞	∞
⅛	45°	4–12	2.00	0.7071	1.414	0.7071	1.414	0.586	1.000	2.000
		14–16	3.00	0.7071	2.121	0.7071	2.121	0.879	1.000	3.000
		20	4.00	0.7071	2.828	0.7071	2.828	1.172	1.000	4.000
		24–30	5.00	0.7071	3.536	0.7071	3.536	1.465	1.000	5.000
		36–60	7.50	0.7071	5.303	0.7071	5.303	2.197	1.000	7.500
1/16	22°–30′	4–12	4.00	0.3827	1.531	0.9239	3.696	0.304	0.4142	1.657
		14–16	6.00	0.3827	2.296	0.9239	5.543	0.457	0.4142	2.485
		20	8.00	0.3827	3.062	0.9239	7.391	0.609	0.4142	3.314
		24–30	10.0	0.3827	3.827	0.9239	9.239	0.761	0.4142	4.142
		36–60	15.0	0.3827	5.741	0.9239	13.859	1.142	0.4142	6.213
1/32	11° 15′	24–60	20.0	0.1951	3.902	0.9808	19.616	0.384	0.1989	3.978
1/64	5° 37′ 30″	30–60	10.0	0.0980	3.920	0.9952	39.808	0.192	0.0985	3.940

This table is useful in field and office for pipe layouts to pass beneath or over sewers, other conduits, streams, etc., or around obstacles.

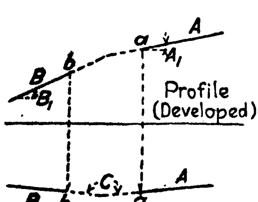


FIG. 182.

Laying Out Combined Pipe Bends. C. A. Jackson[22] develops following formulas: "Profile plane" designates vertical plane through center line of either pipe. Figure 182 shows plan and profile of two pipe lines A and B, to be connected by curve, whose tangent distances are represented in plan and profile by dotted lines.

Let A_1 = angle of inclination from horizontal of pipe A.
B_1 = angle of inclination from horizontal of pipe B.
C = horizontal angle between "profile planes."
D = angle of special in plane of bend* (90 deg. for one-fourth bend).
L = angle between plane of bend and horizontal plane through horizontal diameter of pipe at b.
M = angle between plane of bend and horizontal plane through horizontal diameter of pipe at a.
Y = chord of special = distance from a to b, measured in plane of bend.
X = distance horizontally from a to b.

* Plane of bend is determined by two intersecting center lines of pipe.

Z = difference in elevation between a and b.

$$Y^2 = X^2 + Z^2. \tag{1}$$

$$\cos D = \cos A_1 \cos B_1 \cos C + \sin A_1 \sin B_1. \tag{2}$$

$$\cos L = \cos A_1 \sin C \div \sin D. \tag{3}$$

$$\cos M = \cos B_1 \sin C \div \sin D = \cos L \frac{\cos B_1}{\cos A_1}. \tag{4}$$

Angle D determines what special fits the given case; angles L and M are necessary in drilling bolt holes in flanges (see also p. 331).

Bends in Cast-iron Pipe by Heating. Lengths of 10-in. class D pipe for Cuba were bent to a minimum radius of 50 ft., without breaking a single pipe, by fire beneath cradle of old rails shaped to proper curvature.[23]

Testing Pipe Laid. Plugs may be obtained from pipe foundries, the latest design having coarse threads cast on surface and on face. A plug is calked into last bell in usual way, and can be removed by engaging the lugs with a crowbar, and unscrewing the plug. Holding power of the lead in retaining the plug was investigated at Seattle[24] with results shown in Tables 113 and 114.

Table 113. Dimensions of Plugs, in Inches

Nominal size (diam. S)	20	16	12	8	6
D (spigot diam. of plug)	$21\frac{7}{8}$	18	$13\frac{3}{4}$	$9\frac{7}{8}$	$7\frac{5}{8}$
L (thickness of plug)	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{3}{4}$
M (width of groove in socket)	$1\frac{3}{16}$	$1\frac{3}{16}$	$1\frac{1}{8}$	$1\frac{1}{8}$	$1\frac{1}{8}$
F (diam. of socket)	23	$18\frac{1}{2}$	14	10	8
H (depth of groove in socket)	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{4}$
Depth of lead	$2\frac{3}{4}$	$2\frac{3}{4}$	$2\frac{1}{4}$	$2\frac{1}{4}$	$2\frac{1}{4}$
Annular lead space	$\frac{13}{16}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{7}{16}$

Table 114. Tests of Holding Power of Lead Joints

Size of plug, in.	Load at which plug started, lb.	Area of plug (diam. D), sq. in.	Pressure on plug, lb. per sq. in.	Circum. area, P, between lead and plug, sq. in.	Pressure on circum. area, P, lb. per sq. in.	Depth of lead, in.
6	9,850	45.7	215.5	53.9	182.7	$2\frac{1}{4}$
8	13,500	76.6	176.2	69.8	193.4	$2\frac{1}{4}$
12	17,100	148.5	115.3	97.2	175.9	$2\frac{1}{4}$
16	29,750	254.5	116.8	115.5	191.3	$2\frac{3}{4}$
20	42,000	375.8	111.7	186.2	225.5	$2\frac{3}{4}$
24*	46,250	564.1	81.9	236	195.9	$2\frac{3}{4}$
30*	56,875	865.7	85.1	291.2	195.3	$2\frac{3}{4}$

* Loads interpolated.
The speed of the testing machine head was 0.14 in. per min.

When lines are tested, before backfilling, under an excess pressure of 50 per cent. over maximum expected, and leaky joints recalked, leakage may be reduced to less than 100 gal. per day per mi. per in. diam. Without this precaution leakages as high as 3000 may be expected, and 5000 is not unknown.[25] Burns and McDonnell accept 100 gal. under 100-lb. pressure[25] (see also p. 427).

Filling a Main or System.* Best method of filling a gravity piping system is to open all air valves and blow-offs and a sufficient number of hydrants. Then the valve at the reservoir is opened sufficiently to fill the pipes in a predetermined time. As soon as the water flows full at blow-offs successively, they are shut until water appears at hydrants; finally, the air valves are closed. No water-hammer is experienced, and system is filled in less time than by any other method.[26]

MAKING JOINTS IN BELL-AND-SPIGOT PIPES.

Molten lead has been used as joint material for generations, but is being replaced by Portland cement and compositions like Leadite. Commonly jute yarn or oakum is tightly packed with a yarning tool and hammer into the inner part of the bell, leaving about 2 in. to be poured full of molten lead. This packing prevents lead from flowing into the pipe. To hold the molten lead at the face of the joint, a roll of moist clay has commonly been used. The Star pipe jointer† and similar devices are preferred by many; they are made for each size of pipe, of fabric packing reinforced by flexible metallic bands, with a gate, or opening, for pouring, and a clamp. They should be kept moist while in use.‡ Fig. 181A, p. 407 illustrates equipment for making lead joint.

Calking is done, on small work, by hand, and on large work by air hammers, with a portable compressor driven by a gas engine. Shrinkage of poured lead away from the iron allows leakage; calking reestablishes contact. Calking is effective but ½ in. from the face. See tests of joint, *E. C.*, Dec. 13, 1916, p. 528.

Table 115. Weight of Cast Lead and Yarn per Joint in Cast-iron Pipe[27]

Diam. of pipe, in.	Depth of lead					
	2 in.		1½ in.		1 in.	
	Lead, lb.	Yarn, lb.	Lead, lb.	Yarn, lb.	Lead, lb.	Yarn, lb.
4	6.32	0.251	5.01	0.329	3.16	0.451
6	8.68	0.321	6.88	0.424	4.34	0.579
8	11.03	0.471	8.75	0.605	5.51	0.848
10	13.53	0.578	10.73	0.742	6.76	1.04
12	15.88	0.582	12.60	0.775	7.94	1.05
14	18.67	0.684	14.81	0.912	9.33	1.23
16	21.03	0.642	16.68	0.899	10.51	1.36
18	23.68	0.867	18.78	1.157	11.84	1.56
20	26.33	0.966	20.88	1.288	13.16	1.74
24	31.04	1.137	24.61	1.516	15.52	2.05
30	38.17	1.631	30.37	2.175	19.08	2.94
36	45.28	1.94	36.96	2.58	22.64	3.49
42	68.32	2.92	54.24	3.89	34.16	5.26
48	77.6	3.32	62.20	4.41	38.8	5.97

Width of joint = 0.4 in. for 3- to 14-in. pipe, 0.5 in. above 14 in.

Pipe cutting is required (*a*) previous to laying to fit a given space, remove a defective end, etc., or (*b*) in the trench to remove a broken pipe, to make a connection, or to alter otherwise an existing main. Cutting in trench is

* See also precautions recommended by Finneran in *J. N. E. W. W. A.*, Vol. 34, 1920, p. 281.
† Waterworks Equ[illegible] Co., New York City.
‡ See p. 394 for [illegible] Joints.

done by pipe cutters, of which the French pipe cutter* is typical. Makers claim that pipe can be cut away without breaking or starting next joint. Cutting is also done by oxyacetylene outfits (see p. 346).

Lead Removal. In taking up lines, lead may be removed by melting by fire built beneath the pipe in an enlarged excavation; recent practice favors the oxyacetylene torch. In Brooklyn, a pick-out tool driven by compressed air proved more effective than the torch.[28] Machines are on the market, such as the French lead joint remover.† Methods depend on size of pipe; for sizes above 24 in., fires and other simple methods are not successful.

Leadite,‡ a composition of iron, sulphur, slag, and salt, finely ground and thoroughly mixed, is furnished in 300- to 350-lb. barrels and 50- to 100-lb. bags. It melts at about 400°F., and is poured like lead. Used extensively by Boston, St. Louis, Trenton, Utica, Springfield, and Worcester. Tried in Philadelphia in 1894 and joints reported in 1921 to have required no attention; tried again in 1919 and proved satisfactory; use restricted to places where results may be watched, on account of its inflexibility. Used almost exclusively in Atlantic City since 1903. Pennsylvania Water Co. has used it since 1906 in 4- to 36-in. pipes, up to pressures of 210 lb. In some trenches but 4 ft. deep under heavy motor traffic it has proved satisfactory. For high pressures, Hawley,[29] advises slightly more leadite than specified in Table 116; West[30] reports leadite as running 25 to 70 per cent. beyond this schedule, particularly in 10-, 12-, and 16-in. sizes. The same held true for his work in 1923 and 1924.

Advantages. Calking eliminated; properly made joints will stand 250-lb. pressure per sq. in.; calking bell holes eliminated; pumping reduced to a minimum as leadite sets in water; melts at lower temperature, saving fuel. Melted lead weighs 708 lb. per cu. ft.; melted leadite, 118 lb. Lightness reduces freight and handling charges. Easily poured in restricted space, as in filter gallery; 6-in. space only was available at Charleston, S. C.[31] One ton of leadite will fill more joints than five tons of lead. Pennsylvania Water Co. reported 60 per cent. saving.[31] Bids at Windsor, Conn., in 1917 for a 10-in. line ran 10 cts. per ft. cheaper for leadite than for lead joints.[32] Temperature changes do not start a joint, as with lead. Hawley[29] says that it does not squeeze out under repeated jars. A 12-in. main involved in a landslide suffered but slightly: Used successfully on trolley bridges subject to vibration, as at Boston across Neponset River.[33] A leadite joint can be made in much less time. At New Bedford[104] three men made a leadite joint in 48-in. pipe in 10 min.; lead joint would have required over 100 min. Melting out of joints is easy. Low conductivity mitigates electrolysis effect, but conductivity increases with age. To furnish necessary conductivity for pipe thawing, lead wedges 2 by 2 in. are inserted in each joint, at Springfield, Mass.[33] Tests at St. Louis[34] indicated that cement and leadite joints will stand greater deflections without leakage than lead.

Disadvantages. Sulphur fumes from molten leadite become oppressive in a confined space. Initial leakage is high, but joints tighten in a day or so

* A. P. Smith Mfg. Co., East Orange, N. J. Other makers include Waterworks Equipment Co., New York, Hays Mfg. Co., Erie, Pa., Borden Co., Warren, Ohio.
† A. P. Smith Mfg. Co.
‡ The Leadite Co., Philadelphia.

unless non-rusting water is conveyed. Backfilling is delayed until line tightens to specified leakage. Metcalf and Eddy[36a] found leadite an uncertain material in inexperienced hands; others have found men easily trained to the new material. Joints are less yielding than lead; pipe breakages have occurred in settlements; lead in every second or third joint gives greater flexibility; making lead joints above ground and leadite in trench eliminates bell holes. At Springfield, Mass., a leadite joint leaked due to calking of a near-by lead joint, but tightened in 24 hr.[36b] If leadite does not tighten, pressure must be taken off while leadite is cut out and new joint poured; lead joint would be calked under pressure. Clean spigots and hemp free from oil are essential.

Table 116. Quantities of Lead and Leadite for Pipe Joints[37]

Size of pipe, in.	Lead, lb.	Leadite, lb.	Size of pipe, in.	Lead, lb.	Leadite, lb.
4	8	2.00	18	35	8.75
6	11	2.75	20	40	10.00
8	14	3.50	24	48	12.00
10	17	4.25	30	60	15.00
12	20	5.00	36	75	18.75
14	24	6.00	48	120	30.00
16	32	8.00	....	....	....

Joints assumed, 2¼ in. deep; although 1¾ in. is sufficient for small pipes.

Table 117. Comparative Costs of Lead and Leadite Joints in Cast-iron Pipe[30]

Size, in.	Average joints made per day	Cost per day, crew and material		Cost per joint		Saving per joint
		Lead	Leadite	Lead	Leadite	
6	50	$56.08	$25.85	$1.121	$0.517	$0.604
8	50	66.58	29.10	1.331	0.582	0.749
10	45	70.60	43.55	1.57	0.968	0.602
12	43.8	74.45	46.64	1.70	1.064	0.636
16	32.3	100.63	49.51	3.116	1.532	1.584

Metalium* is a composition of dark, grayish luster, furnished in pellet form in 100-lb. bags, poured like lead, melting at 250 to 300°F. Light weight (one-fifth of lead) and high specific heat lessen costs. Has been used in Omaha since 1914.[35] At Davenport three or four pipes are jointed on sticks laid across trench, and lowered by derrick. No calking. Bell holes may be small. Material costs about 40 per cent. of lead; is less salable, and therefore there will be less thefts. It sets rapidly in place with considerable loss of volume.† Tests by Bass and Jensen indicated that metalium joints are not as tight under deflection as cement or lead.[38] If too high a temperature is used, it thickens so as not to pour readily.[38] This requires constant attendance, with high labor cost.[39] Special jute costs twice the jute required for lead. Tests at St. Louis indicated that metalium cost more in the joint than lead.[39] Initial leakage is high.

* Met[illegible] Omaha, Neb. (Out of business in 1926.)
† Unl[illegible] why is not calking required? (EDITORS.)

Lead-hydro-tite* is claimed by makers to cost 50 per cent. of first cost of lead; to allow smaller bell holes and to eliminate calking. Weighs one-fourth of leads, and has more elasticity. To estimate quantities, for 2½-in. joint space: radius of inside of pipe in in. = lb. of lead-hydro-tite per joint. Minor difficulties attend the first use with inexperienced men and incomplete equipment. Used at Cambridge, Mass., Weymouth, Mass.,[40] Berlin, N. H., and New Bedford. Leakage from 8-in. line under 70-lb. pressure after 1 year amounted to 0.25 g. p. d. per lin. ft. of joint.

Cement (Portland) either neat or as mortar, has been used many years for gas and water-pipe joints. Los Angeles was the pioneer. Used exclusively by the East Bay Water Co., California. Portland, Ore.,[41] uses cement on all pipes, 6- to 30-in., unless demand for service will not allow setting time. Yarn of hemp rope should be used. Pipe must be braced against jar and workmen kept off after joint is made. Common procedure is to calk in one braid of hemp yarn, fill joint within ½ in. with cement filler, then place another braid of yarn, and calk until the cement filler is firmly packed. Joint is then wiped with cement paste, finished on a 45° bevel. Spring Valley Water Co.[40] uses two rings of jute or yarn, free from oil and tar, to start the joint, and cement mixture is so dry that it will crumble in the hand; 1 lb. water to 14 lb. cement. At Long Beach,[42] hammer is not applied for calking until cement has been placed in half the space; joint stood 48 hr. before pressure was put on. One calker made 24 12-in. joints per 8-hr. day. Wastage of 20 per cent. of cement was found at Long Beach. U. S. Housing Corp. used mix of 2 parts cement to 1 part clean, sharp sand.[130] Neat cement has tendency to crack; this shrinkage can be largely eliminated by mixing 2 or 3 hr. before use.† Some gas companies use 3 parts cement and 1 part sand. Detroit City Gas Co. finishes the joint with lead wool.[43]

At Winchester, Ky.,[30] 1 lb. of water to 13.5 lb. cement (7.4 per cent.) mixed in iron pail with trowel. Cement per 12-in. joint, 8.3 lb.; per 10-in., 6.9 lb., waste included. Joint maker wore rubber gloves, stuffed cement paste into joint by hand, and rammed in with calking iron; hammer was not used for first filling; but third to fifth batches were calked with 3½-lb. hammer until little impression was made at face. Average time, joint 2½ in. deep, was 25 min. for five men. When tested up to 160 lb. per sq. in., leakage averaged 0.50 g. p. d. per ft. of joint. Joints were covered with 6 in. of earth until cured.

Cement vs. Lead.[30] Cement has greater compressive strength and adhesion than lead. Its expansion coefficient is nearer that of iron. No temperature changes are imposed on the joint during construction. Cement expands while lead contracts, giving a fuller joint. Cement joints are more difficult to remove than lead. Cement is at a disadvantage in wet trenches and cold weather. Cement joints must stand longer than lead before pressure is put on. Cement and leadite joints will stand more deflection than lead.‡ The great disadvantage to cement is its rigidity under settlements and vibrations; where these conditions occur, Los Angeles substitutes lead joints. There

* Hydraulic Development Co., Boston.

† See also report to American Gas Institute by Cast Iron Pipe Comm., *Am. Gas Eng. J.*, Nov. 8, 1919, p. 444.

‡ See tests at St. Louis, *E. N. R.*, Aug. 2, 1923, p. 190.

will be some initial antagonism of labor. A cement joint costs one forty-eighth of lead for material, but labor costs are equal or greater on account of keeping joint moist during setting.

Lead wool must be calked pneumatically, as hand work is too expensive. No melting required. A lead-wool joint takes two to three times as long as poured lead. It has greater strength. Many gas lines have lead-wool joints (see Simpson, *Proc. Am. Gas. Inst.*, 1910, p. 651; also *E. N. R.*, Aug. 23, 1923, p. 310). Calkers often refuse to work with it. Prices were kept up by patents which have expired. In Springfield 42-in. line proved tight under 150 lb. per sq. in. pressure. Has advantages in certain difficult locations.

STREAM CROSSINGS

Submerged Pipe Lines.* *Pipe Material.* Submerged pipe lines have been laid in all parts of the world, by ordinary methods, of all ordinary pipe materials except wood staves. Universal pipe† was used, at Puerto Barrios, Guatemala,[45] for a 6-in. line, 15,000 ft. long across a bay with maximum depth of 24 ft. Cradle used, reaching bottom at slope of 14 deg. Cost of laying, 22 ct. per ft. The longest line (1926) is the intake of waterworks at Burlington, Vt., 15,480 ft. long, laid on the bottom of Lake Champlain. Maximum depth, 120 ft., occurs at New Orleans in Mississippi River. Pipe is usually ordinary cast iron, with the coating common to street mains. Special hard cast-iron, riveted steel, riveted wrought iron and ordinary wrought iron have been used, usually coated in the ordinary manner, but in special cases protected by concrete shells, bedded in concrete, or lined with brick. A cast-iron pipe in Boston, in service 50 years, was found badly tuberculated inside; in fair condition outside where covered but where uncovered the iron was soft; in places it could be cut to some depth by a knife (graphitic corrosion). One cast-iron pipe carried salt water 20 years without sign of failure.

Joints. All ordinary types of rigid joints have been used, the common bell-and-spigot type, run with lead, most frequently. Every fifth or sixth joint usually is modified by turning the spigot to a slight taper. The tapered joint is made up with lead on shore, a clamp or strap retaining the lead; the spigot is withdrawn and reentered under water, where a diver calks the lead. Screwed joint of wrought-iron pipe is least satisfactory of rigid joints, due to shearing of threads. Flexible joints are used to fit irregularities of the trench and avoid straining. Majority are ball-and-socket joints with bearing surfaces of lead on iron. All-iron ball-and-socket joints have been used with ordinary gaskets. These joints allow deflections of 6 to 17 deg., usually 10 deg. Flexible jointed cast-iron pipes are usually made in 12-ft. lengths. Possibly there is economy in longer pipes due to weight of bell, but, with small demand for such pipes, is apt to be more than offset by the greater cost of extra-long pipes. Shorter lengths are better in some cases to make a more flexible line, or to make lighter pieces to handle. Usually, flexible joints are introduced only at intervals of three to six lengths, other joints being rigid,

* See also "Designing [illegible] Flexible Joints for Submerged Water Pipe Lines of Cast Iron and Steel," E. Kuichling, *Eng. [illegible]*, Oct. 15, 1914, p. 432; also inset, p. 418.

† See 394 [illegible]

for maximum economy. Steel pipe of large diam. is laid in long lengths, with flexible joints at intervals of 100 to 116 ft. Here connection is not made at a flexible joint, but near it, by a flanged or hub-and-spigot joint. Length of section is usually determined by method of laying.

Foundation. It is usually desirable to cover the pipe after laying; this prevents displacement by currents, flotation, damage from navigation, and increases the durability. Trenches are dredged or washed out. Sometimes it is necessary to put in foundations to prevent settlement where a bottom is soft. Series of pile bents with caps on which the pipe rests and is strapped, wooden blocks laid on bottom of trench, excavating to greater depth and refilling with sand or gravel, strapping timber platforms to pipe in case of steel pipe, and concrete foundations have all been used. The trenches are allowed to silt full, unless danger from anchors calls for prompt refilling. If jeopardized by currents or flotation, refilling is done with care, with various means of anchorage. Pipes have been sunk in mud or sand, after laying, by a water jet or by scouring of the current under the pipe, held just clear of the bottom.

Laying. (*a*) One method of laying pipe is from a construction trestle, built over the trench. The pipe is lowered simultaneously at all points of support, if it has rigid joints. Otherwise, as a joint is completed, it is lowered from the trestle, the other end remaining suspended. This method is not suitable in rough or deep water, or where navigation must not be obstructed. Used for 30-in. pipe at Des Moines.[46] (*b*) Pipe is sometimes laid on ice and lowered by tackle through an opening in the ice. (*c*) With suitable joints, a pipe can be lowered and connected under water by divers. The pipe is made up on shore in sections, which are floated out over the trench. Flotation is accomplished with casks, through the buoyancy of the pipe when bulkheaded, or it is lowered from a scow. After being connected by the diver, the joint is pulled home by special hydraulic jacks or other means and calked if necessary. (*d*) With flexible jointed pipe* in deep water, it is customary to lay and sink the pipe continuously, joints being connected on a scow, and launched from the stern through a chute. The end is fastened securely to the scow. After pouring, each joint is deflected sufficiently to break the adhesion between the lead and iron before launching. Care must be taken that the suspended line is not deflected so as to wedge the bell end against the body of the pipe; this can be guarded against by setting the launching ways so that the pipe will "rise from it," unless the safe angle of deflection has been exceeded. In the suspended line, the greatest deflection is at the bottom. This method insures considerable tension in all joints, drawing them tight, but it is difficult in rough weather. Rods running the length of a section are often used to give rigid joints sufficient tensile strength. (*e*) The pipe is also laid in a slide or cradle extending from the scow to the bottom of the trench, built on a curve corresponding closely to shape assumed by pipe line hanging freely and each joint fully deflected. The pipe slides on rails. A plate at the bottom of the slide prevents sinking of slide in trench. Slide may be suspended between scows, or through a well hole in or from one end of a large barge; it is adjustable, s[illegible]he curve of the

* Ordinary joints were used at Galveston, Tex.[47]

suspended pipe will be tangent to trench bottom. Stern anchors are necessary, due to the forward thrust caused by the weight of the pipe. (*f*) The whole pipe line may be built on shore and hauled into position, buoyancy of pipe or auxiliary floats being used. When dragged along the bottom, in swift water, a conical pilot is fixed on the first section. Hauling is done by a cable attached to the first section, or run through the pipe and attached to the last length, and to a winch on the farther shore. In still water the pipe can be floated to position and sunk, or pushed from shore. Pipes of all types of joints can be laid this way, except in heavy currents (see Table 118 for data on these methods). At Waco, Tex.,[48] 10-in. gas mains were welded into 100-ft. lengths and pulled out on falsework (550-ft. crossing) and welded to neighbors. Whole lowered by block and tackle.

Testing. Submarine mains are tested after completion by filling with water under pressure and measuring the leakage by meters. Sections may be tested as completed. With flexible joints, this method tightens the line. Tests with compressed air are also made; the escaping bubbles aid in locating the leaks, if any. Leaks are discovered by divers and calked. Tests show that submerged lines can be made as tight as practical purposes require. On Bayonne, N. J., line (24-in. diam.) leakage limit was placed at 1 cu. ft. per min., and it tested far below this. The 16-in. line at Portland, Ore.,[49] 1007 ft. long, was specified to have a leakage limit not above 16.8 gal. per joint per 24 hr. It was actually accepted when leaking 60 per cent. in excess of this; several months later, tests indicated negligible leakage. The first Narrows siphon, New York City, tested immediately after completion, under pressure of 130 lb. per sq. in., leaked 5.5 g.p.m.; a 40-day test, some months later, under pressures of 100 and 110 lbs., showed leakage of but 0.75 g.p.m.

Pipes in Salt Marshes. Cast-iron pipes (12-in.) across salt meadows, Atlantic City, have been down 19 years. Action of meadow mud is very severe; after 18 months in it, wrought iron looks as if it had been in acid bath. Deterioration in 19 years from half to full thickness. This was due to presence of sphagnum or bog moss, which has acid-producing powers; sometimes classed as peat.[51] During severe winter, 1904–1905, this main froze for first time and 22 lengths broke, probably due to weakness from corrosion.[52a] The 36-in. cast-iron pipe laid in 1863 just above surface of salt marshes along Passaic River, and supported on piers with no earth covering, still showed makers' mark and date in 1914, "W. F. & M. C., 1862" (Warren Foundry & Machine Co.).[52b] Wood pipe is best to carry water through salt marshes or salt water. Salt has no decaying effect upon wood, and if the pipe is wound with copper wire it will last much longer in salt water than cast iron or steel. A 48-in. wood-stave main was laid (1911) across 7 mi. of salt marsh at Atlantic City, N. J., to replace a steel main laid in 1901, badly pitted, and the cast-iron main, laid in 1882, also pitted in spots. The wood main has staves 2 in. thick, banded with wrought iron, $\frac{5}{8}$ in. thick, for a pressure of 75 lb. Long Beach and Elmhurst, L. I., also have wood pipes through salt marshes; the latter is 20 in. in diam., 2000 ft. long, operated under pumping pressure.[53] See also pp. 358 and 81 Horizo... Con... 394...

Wooden pile bents have been used to keep pipe above surface. The muck has timber-preservative properties. At Wilmington, N. C.,[54] pipe was placed so that 8- by 8-in. creosoted cap was 1 ft. back of each bell.

Concrete can be used successfully to prevent the corrosion of metallic pipes; 1:2:5 mix, using sand and gravel, densely packed, 4 to 5 in. thick. Joints in the concrete envelope to provide for expansion and contraction should, in latitude of Middle Atlantic States, be about 30 ft. apart. A thin copper sheet, about 4 in. wide, should be inserted in the concrete at the joints, to prevent leaks. Concrete of ordinary Portland cement concrete has generally been injured by sea water and even sea moisture in course of 10 years. (See Report National Research Council, Marine Piling Committee, and Atwood & Johnson, *T. A. S. C. E.*, Vol. 87, 1924, p. 204.)

Water-main on bridge.* The successful design[55] shown in Fig. 183 overcomes effects of bridge expansion on pipe laid directly on bridge, avoids

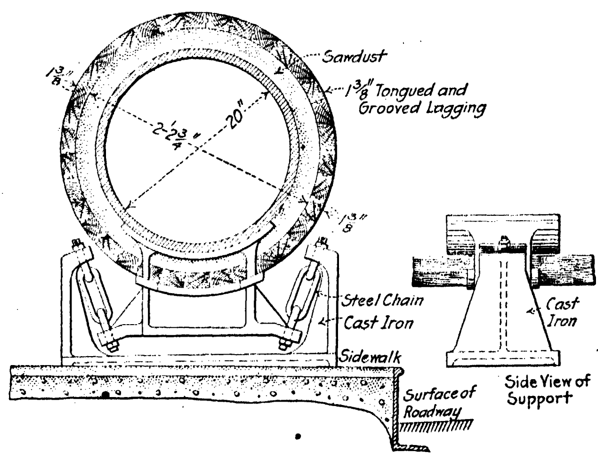


FIG. 183.

expansion joints, and protects against freezing and vibrations. Vertical or 45° riser should be used where pipe rises above ground. Anchor by bolting through lugs on bell, or using concrete. Vibrations on bridge loosen lead joints; leadite has given satisfaction.† Provisions against freezing are essential.‡ Two boxes with an air space between should enclose small pipes on bridges; one box filled with tan bark is not sufficient protection in New England.[1d] Occasionally, special bridges are built, as at Great Lakes, Ill.,[121] or the bridge of 36 spans (107 ft. each) across the lower end of San Francisco Bay for the Hetch-Hetchy conduit. Standard wrought-iron pipe and fittings (4 and 6 in.) were used for 6 years on Pecos Viaduct without any trouble from vibrations.[125]

MAINTENANCE AND OPERATION

Wet Connections. Connections to cast-iron mains under pressure are made by special tools without interrupting service. A two-part sleeve is

* See also "Pipe Lines on Bridges," Brockway, Public Works, Vol. [illegible] 1924, p. 239.
† See p. 413.
‡ Ice formed on leaky pipe caused failure of a bridge at Utic

bolted to the main at a point between joints where the connection is desired and joints of sleeve calked with lead or made tight by gaskets. One part of the sleeve has a flanged outlet; to this a permanent flanged valve is bolted. To the other flange of this valve is bolted temporarily a cast-iron dome containing special cutters for cutting from the main a disk of the diam. of the proposed connection and a combination drill and tap, carried by an axial spindle which projects through the stuffing box in the dome. For connections up to 24 in., hand operation with ratchet levers is used; for larger sizes the machine is driven by a small engine or motor. Valve being open, first drill a hole through the center of the connection and then engage with the tap so as to hold the disk when cut. After the disk has been cut and withdrawn through the valve, valve is closed, dome removed, and permanent pipe bolted to valve. Great care is necessary to start the drill straight, otherwise it may be broken; must not feed too fast or the drill may be broken or pipe cracked, and it may be impossible to remove the disk after cutting. Stuffing-box gland in dome should be loose to permit escape of air until water fills dome. For larger connections, after preparations are complete, about 1 day is needed; smaller connections may be made in half day. Connections from 4 to 36 in. have been made by the A. P. Smith Mfg. Co., Newark, N. J. Waterworks Equipment Co., New York, build an engine-driven pipe-tapping machine for making connections up to 48-in. diam. in the dry. Tapping machines for services are made by Hays Mfg. Co., Erie, and Mueller Mfg. Co., New York.

Pipe-line Raising. Due to regrading at Seattle, it became necessary to raise a 20-in. cast-iron main under 130-lb. pressure 17 ft. above its old location, without interruption of flow. Center of 1800-ft. length was raised this amount, the ends remaining in old positions. Jacks and cribbing were employed. A 60-ft. section would be jacked up 1 in., men working to signal to give even raising. Then the adjacent 60-ft. section would be raised same amount, cribbing placed beneath, etc. Practically no leakage occurred at joints, although some had to be recalked occasionally.[56]

Air and Sediment. The capacity of a pipe line may be seriously diminished by the accumulations of air at summits, and of sediment in depressions (see *J. N. E. W. W. A.*, Vol. 34, 1920, p. 280). Air relief valves should be placed at summits and blow-off valves at depressions. See also Air Valves, p. 446.

Welding and electric torch have been employed in repairing breaks (see p. 347). A broken section of the Narrows siphon, New York, was burned out by electric torch under 50 ft. of water.[128]

Sleeves split for ready insertion in an emergency, *e.g.*, a cracked pipe, are made by Water Works Equipment Co., A. P. Smith Mfg. Co., and others. Sleeves are also made to fit over cracked bells; their use is claimed to save much time in repairs, and involves less trouble and expense than insertion of new pipes.

Freezing. Pipes freeze more easily in dry ground; in wet ground the latent heat of the moisture retards frost. Freezing depends also on exposure and on velocity of flow, the pressure being of negligible effect. Subaqueous pipes in salt water have frozen, and pipes have frozen during a thaw (the reasons are outlined [illegible] in *J. A. W. W. A.*, Vol. 3, 1916, p. 966). Tests at Portlan[illegible]ea water reaches 28°F. in winter, indicate that

pipes larger than 6 in., in salt water, will cause little trouble if buried 6 in. in blue clay.[49] Pipes in rock trenches, refilled with rock, are less likely to freeze than in moist, compact soil. An air space of "dead air" around pipes reduces freezing tendency. Dead ends should be avoided and circulation made as free as possible.

In freezing, water expands about one-twelfth of bulk, and exerts an expansive force of about 30,000 lb. per sq. in. when rigidly confined, and about 10,000 lb. in pipes. Many weak pipes burst under these conditions. A 6-in. cast-iron pipe laid in 1892, at Springfield Mass., by regrading of a street was left with but 27-in. cover.[57] Small flow in recent years accelerated freezing. Freezing in 1912 broke a pipe length; ice was found solid throughout length, except for a cylinder 1 in. in diam. about 1 in. above invert; probably this was last part to freeze. All iron rust and tubercles amassed during 20 years service were concentrated around this 1-in. opening; no rust was left on the periphery. The winter of 1917–1918[58] caused the greatest trouble from frozen pipes. The A. W. W. A. had a committee report on this cold period (see *J.*, Vol. 7, 1920, p. 749). Temperature at Savannah, Ga., dropped to 14°F., and E. R. Conant[59] estimated number of broken pipes at 10,000.

Thawing has been accomplished by fires, steam, and electricity.* The first two methods involve uncovering the pipe; although costly and slow, they have the merit of disclosing breaks. It is impossible to use electric thawing where there are a large number of composition joints, unless lead wedges are placed in the joints.[60a] (see p. 413). Insulating for electrolysis also interferes with electric thawing. Current is used from lighting wires, portable motor-generator sets, or storage batteries. In first system, high-tension overhead primary wires, carrying 2200 to 2400 volts, were tapped, and fuse boxes and transformers of 15 to 75 kw. were put on distribution line; voltage reduced to 110 and amount of current delivered upon pipe was further controlled by a water rheostat.[62] The Committee of N. E. W. W. A. prefers the motor-generator,† set as it is flexible and portable, can work continuously, can be made nearly foolproof, does not deteriorate when idle, and is least liable to endanger the pipes. Middletown, Conn.,[61] equipment mounted on 1-ton truck: 25-kw. transformer for stepping down 2300-volt a.c. current to 50 volts, an instrument board, and a water rheostat, simply a barrel filled with salt water, in which were immersed two flat coils of heavy copper wire, mounted on adjustable supports for controlling the current. Mains up to 6-in. were thawed. Boiling of the rheostat was reduced by putting in snow.

Services are often thawed by running into the service from the cellar $\frac{3}{16}$-in. block-tin tubing, through which hot water is pumped continuously by a force pump. Tubing cannot be pushed far through $\frac{5}{8}$-in. services.[60b]

Cleaning water mains‡ removes incrustations, growths, and sediment, all of which decrease capacity. Mains may be cleaned by flushing, by pulling through a cleaner, or by water-driven turbine cleaners. Latter are used by National Water Main Cleaning Co., New York. For methods used in

* See *Bull.* 7, Purdue Univ. Expt. Sta., 1924.

† For motor-driven generator used at Dedham, Mass., see *J. N. E. W. W. A.*, Vol. 34, 1920, p. 114.

‡ For other discussions, see following volumes of *J. N. E. W. W*[illegible]*al*, Vol. 5, 1891, p. 21, 131; Vol. 24, 1910, p. 373; Vol. 28, 1914, p. 70. *St. Johns, N. B.*, V[illegible]3. *Boston*, Vol. 13, 1899, p. 341.

Great Britain, see *E. N.*, Aug. 31, 1911, p. 259, and *Munic. Eng.*, 1916, p. 191. A questionnaire on American practice is given in *Munic. J.*, Aug. 10, 1916, p. 152, and June 15, 1918, p. 486.

Flushing consists in opening a number of hydrants or blow-offs. This is liable to injure street pavement, to overload sewers, and so to clog meters with sediment as to cause trouble.[63] Detroit experience is that flushing removes loose dirt, but barnacles and solidified sediment on bottom remain.[64]

Flue Cleaner. Railway water lines carrying softened water require frequent cleaning. A line on the Northern Pacific[65] has been cleaned by making pits 8 by 16 ft. at 600-ft. intervals where pipe length could be taken out; a line of sewer rods is pushed through carrying $\frac{3}{8}$-in. cable, which pulls through a large-size flue cleaner. This was used on 17,000 ft. of 10-in. line at Dilworth, Minn.

Cleaning by machine consists in cutting out a pipe length at each end of section to be cleaned. The downstream section is replaced by a $\frac{1}{8}$ bend, and a riser of the same size is extended above the street. The upstream section is replaced by a length of new pipe, containing the "Go-devil," which is calked into place. Water is then turned on above, sending the sediment and the "Go-devil" eventually out of the riser. Cleaning by machine offers the incidental advantage of checking the records of valve location. The "Go-devil" may be drawn by cable. Cleaning may be so scheduled that a main is out of service but 12 hr.[67] At New Albany, Ind., a 2-mi. stretch of 16-in. pipe was traveled in 40 min., and two 5-ton truck loads of rust tubercles brought out.[124]

Service pipes are cleaned in Springfield, Mass.,[69] by inserting jointed $\frac{1}{4}$-in. brass rods from cellar end, carrying a double-edged knife at forward end.* Stuffing box prevents great loss of water, as pressure must be on for the best results. Value to consumers reported as $15,600 in 1 year.

Results of Cleaning. Some pipes at St. Louis[70] tested before cleaning indicated 33 per cent., and after cleaning 96 per cent capacity of new pipe; deteriorated in a few months to 65 per cent., a stable condition. Examinations of 6-in pipe showed coating abraded and rust streaks forming. The springs of the machine should be so set as to insure thorough cleaning and yet not injure the interior coating. Mains 20 years old will generally show large improvement in capacity after cleanings. Cleaning is particularly advisable on long conduit lines or where pumping costs are high.[71] See Hodgman, *Proc. A. W. W. A.*, 1911, p. 40. Incrustations during first year after cleaning may produce a noticeable effect but not after that; Hodgman cites this as the experience in Cincinnati, Buffalo, Boston, Cambridge, Shreveport, Kansas City, and Omaha.[72]

Hartford, Conn.[66] 33,100 ft. ($6\frac{1}{4}$ mi.) of 20-in. and 30-in. mains were cleaned in 49 days, using 1,655,000 gals. of water. Some incrustations 1 in. thick, nearly around pipe. Some pipe moss (Paludicella) found near reservoir. List prices of National Water Main Cleaning Co. for doing this work in 1912† ranged from 16 cts. per linear foot of pipe for 6-in. pipe, up to 80 cts. for 36-in. Carrying capacity was increased from 50 to 61 per cent.

* "Sta[illegible]

† 1925 [illegible] 20 cts. for 6-in., up to 70 cts. for 36-in., for favorable condition.

Economy of cleaning lies in reduction of pumping charges and the maintenance of fire pressure. Ledoux[73] cites a 6-in. pipe, 25 years old, the capacity of which was quadrupled. Alternative to occasional cleaning is to treat the water so thoroughly as to free it of all constituents which cause incrustations and growths; few instances of success are known; see p. 387 and p. 813. Charleston, S. C., is avoiding excessive cost of cleaning by laying cement-lined pipe. See *E. N. R.*, Sept. 7, 1922, p. 387.

DISTRIBUTION LOSSES

Water unaccounted for is defined by A. W. W. A. Committee on Water Waste Control[75] as "that portion of the water flowing into a distribution system which is not delivered to the consumers." It embraces (*a*) leakage, (*b*) waste, (*c*) underregistration of meters, (*d*) water used for sewer flushing and elsewhere not metered, but estimated. Holway[76] reports that in Oklahoma City the greatest losses were through (*a*) illegal connections, of which an 8- and a 3-in. were found, and a bypass on a 1-in. meter; (*b*) leaky flushing tanks in sewer system; (*c*) poorly maintained meters.

In systems 85 to 100 per cent. metered, usual quantity unaccounted for is 20 per cent., exclusive of pump slippage. Committee on Water Consumption of N. E. W. W. A. (1913) considered 25 per cent. loss in a well-metered system as good practice. N. E. Committee on Meters (1916) found a range from 12 to 49 per cent., with average 27, for 35 completely metered systems.[77] At Grandview, Ohio, water unaccounted for in summer was 3.5 times that in winter. Huy[12] was able, by pitometer surveys, overhauling meters, and a house-to-house inspection, to increase revenue-producing quantity from 48 to 90 per cent. of total pumped. See Fenkell on Detroit waste in *J. A. W. W. A.*, Vol. 8, 1921, p. 583.

Water waste comprises water which serves no legitimate purpose, and includes leaks. Sources of waste which involve the human factor are: (1) failure to turn off spigots, (2) failure to turn off hose, (3) unnecessary sprinkling of lawns and sidewalks, (4) leaky fixtures, (5) toilets with too large tank capacity, (6) deferred maintenance of fixtures, (7) unauthorized use of water other than from fire-protection systems (for which a readiness-to-serve charge only is made), (8) hydraulic elevators not reusing the water, (9) street and sewer flushing, (10) unauthorized use of fire hydrants for steam rollers, etc.[78] Some items listed are not entirely waste, although commonly attended with waste; some may result in loss of revenue through a faulty system of charges. Cantonment regulations for leakage and waste tests are given in *J. A. W. W. A.*, Vol. 6, 1919, p. 179.

Water Wasted by Fixtures. A $\frac{1}{16}$-in. stream leaking through a worn faucet washer under 40-lb. pressure will discharge 19,000 gal. in a month; at 20 cts. per 1000 gal., this would pay for a new faucet* in a month. A $\frac{1}{4}$-in. stream, as from an overflowing toilet flush, will waste 300,000 gal. per month; this would equal the cost of a toilet complete.[79] Publicity measures for curtailing waste often feature this phase. In New York City a circular† was distributed showing faucets at various openings, labeled with annual

* Exclusive of plumber's charges (Editors).
† Reproduced in *J. A. W. W. A.*, Vol. 6, 1919.

expense to householder, from $2 to $800. True[80] estimates that 12 to 20 per cent. of total water consumption is used in toilet fixtures, and that this can be restricted by properly designed fixtures without violating sanitary requirements. Use was restricted in cantonments by bending down the floats in the flush tanks so as to reduce discharge. Schools waste enormous quantities. Boys' High School, Louisville, wasted 114,000 g. p. d.[81]

Waste-water surveys establish relative flows in districts by sectionalizing the distribution system. Comparisons of flows* between midnight and 5 a.m., with daily average, indicate excesses which should be investigated in detail by reducing the size of district or by use of leakage detectors, or house-to-house inspection.

Pitometer surveys, developed from the Deacon system used in England, require knowledge of the areas of the waterways (net interior cross-sections of pipes), and determination of velocity by some form of pitot tube, introduced into the pipe through a corporation cock.† See Lanham, *J. N. E. W. W. A.*, Vol. 33, 1919, p. 287. Pitometer surveys are made at least expense where system contains manholes at strategic points with 1-in. corporation cocks on either side of the valve. A successful pitometer survey requires training of men.[82] The pitometer is reliable at velocities as low as 0.5 ft. per sec. Discharges may be determined by pitometer within 2 or 3 per cent.[83] Pitometer surveys yield varying results. At Detroit, it was estimated that a saving of 20 mgd. would be effected (15 per cent.), whereas a survey of but half the system revealed only 7 mgd. The saving possible is difficult to estimate in advance.[75] A pitometer survey at Madison, Wis.,[84] costing $4600, disclosed leaks on which the pumping charge for 1 year was $7000. Water-waste survey costing $5200 (total, including city time) saved Newark, Ohio, about 2.1 mgd.[85]

Hose-and-meter survey[82] is adaptable to small systems where velocities are low. A meter is installed on a hose line between two hydrants on opposite sides of a valve. The valve is closed and the hydrants opened, bypassing all flow through the meter. Comparisons of flows can thus be made as desired. Meter sizes $\frac{3}{4}$ to 2 in. have been used. Two-inch Venturi meter with $\frac{5}{8}$-in. throat was employed at Oak Park, Ill., to detect low flows.[86]

Pressure gages may be used.[87] Sectionalize a part of the system and establish a pressure gage on its feeder. Close valve on feeder between gage and pumping station or reservoir. If at hours of small consumption pressure on gage falls rapidly to zero, leaks are indicated.

House services were tested at Oak Park, Ill., by inserting pitometer. Flows as low as $\frac{1}{4}$ g.p.m. were measured.[88] Autographic detectors were also used.

Test-pit method[89] requires a manhole built around the valve, so that corporation cocks on the pipe on each side of the valve are accessible, making possible a temporary bypass with meter inserted. Close the valves at the ends of the section to be tested and open an intermediate hydrant; "crack" one valve until water rises to the lip of the steamer connection. Then close the valve; if flow from nozzle continues, the valve is leaking. If level in

* For test method [illegible] W. W. A., Vol. 12, 1924, p. 157.

† Equi[illegible] Pitometer Co., New York, Simplex Valve & Meter Co., Philadelphia, and [illegible] Co., New York.

hydrant drops, there is a leak in mains. With all gates closed, the only entry is through the meter; by shortening the length between test pits, the leaky section can be determined, and the leak ultimately found by means of surface indications of moisture, aquaphones or other means.

Value of Water-waste Surveys.[90] *Boston,** as a result of 3 years' work, found 296 leaks, including two broken mains, accounting for 10.25 mgd. in daily consumption of 85.1 mgd. Survey cost $68,000. *Grand Rapids* survey began 1921, cost $10,000, disclosed no large breaks or leakage, but proved valuable in tracing dual piping system of which no record existed. *Detroit* survey covering 1151 mi. of 24-in. main (and under) disclosed underground leakage of 9.6 mgd., including partly open 6-in. blow-off and two breaks in 6-in. main. Test of large meters in place showed underregistration 0.43 mgd., worth $51,000 annually. House-fixture leakage amounted to 22.6 mgd. Survey located many defective valves. Department has established pitometer division as result of waste survey. *Baltimore* survey began 1920, covering 153 mi. of main, reduced consumption of 30.9 mgd. by 6.5. Half of this amount was found inside curb cock. Cost of survey averaged $168 per mi. of main. *Herkimer,* N. Y., survey costing $1200 disclosed no considerable leakage, but cost was considered justified by determination of pump performance and of defective valves and hydrants. *Ogdensburg,* N. Y., with consumption of 3.07 mgd. avoidable waste, including underground leakage of 0.35 mgd. due to two joint leaks, one cracked main, and 23 service leaks. *Elmira,* N. Y., found survey profitable, although leakage not stated. *Oswego,* N. Y., survey accounted for 1.0 mgd. lost through leakage on unmetered fire lines. *Richmond,* Ind., survey disclosed underground leakage, 0.27 mgd., and house wastage of 0.37 mgd. Water saved was estimated to pay for survey in 1 year's time.

Leakage Detectors. Aquaphone, geophone, sonograph, sonoscope, sonofone, Darley leak locator,† and detectaphone are trade names for appliances, the principle of which is the audibility of water flowing through a leak. All but geophone and Darley leak locator require direct contact with the pipe. Darley detector rests on the ground; high wind or other noises interfere.[91] Clark leak indicator operates through loss of pressure; it is used for services only. Leaks may be located by water-pressure diagrams.‡ Leaks were detected at Madison, Wis., by bare spots when ground was snow covered.[84] A leakage of 650 g.p.d. per mi. seldom softens the ground,[92] or gives surface indications, unless concentrated in one or two leaks. For locating pipes, the detector,§ wireless pipe locator,‖ and similar devices are available.

Aquaphone. Sounds of leaks investigated by aquaphone in New York decreased rapidly with increase in size of main. Leaks of 0.5 mgd. on a 48-in. main could not be heard beyond 10 ft., while smaller leaks on 6-in. pipe could be heard 500 to 1000 ft. distant.[93]

Geophone[94] embodies principles of seismograph, a lead weight being suspended between two plastic diaphragms across small airtight box. Any vibration will affect the tight box and compress or rarefy the air, which effect is transmitted to the earpiece resembling a stethoscope. To detect earth sounds,

* See also McInness in *J. N. E. W. W. A.*, Vol. 35, 1921, p. 34.
† W. S. Darley & Co., Chicago.
‡ Force pump was used on services at Superior, Wis.; see *E. N.* [illegible] 17, 1921, p. 461.
§ Water Works Equipment Co., New York
‖ Modern Iron Works, Quincy, Ill.

the instrument must rest on the ground. Trial positions with two geophones with an earpiece to each ear will establish point of maximum sound. At Roanoke, Collins[95] found that time taken to locate a leak when within 50 or 75 ft. of it usually amounted to a few minutes, not exceeding 15 or 20. Survey cannot be made during traffic hours, as all noises are detected; best time, except in outlying districts, is early morning. Operator must learn to distinguish sounds. Sounds of leaks vary with the impinging medium.

Waste control on premises* may be had by house-to-house inspections† or by metering. Inspections must be frequent, are costly, never completely effective, particularly in large cities, and are resented by householders[96] whose cooperation is thereby forfeited. To be even measurably effective, they must be accompanied with rigid imposition of penalties. Metering‡ produces permanent results; costs about the same. Notwithstanding prejudices, it is noteworthy that a municipality that has adopted meters has never gone back to the old system.[97] Waste is cut down by reduction of pressure either at pumps or by throttling main-line valves.§ Hartford saved 0.3 mgd. by throttling.[98]

Water-waste control, in Oak Park, Ill.,[99] a city of 35,000, is attained by annual waste surveys, complete meterage including all public uses, maintenance of meters, rigid collection of high bills caused by fixture leakage, education of consumers, and absolute backing by the higher municipal authorities.

Leakage from Mains. Probably leakage in a new system will not be materially less than 3000 gal. per mi. daily unless carefully tested and all defects remedied. Leakages are proportionately greater in small-town systems, as many of them are poorly constructed. By testing in open trench, under pressure at least 50 per cent. in excess of maximum static pressure expected, recalking all dripping joints and replacing defective pipe, it is possible to reduce leakage to an extremely low amount. With first-class calking, when the men know the work is to be tested in open trench, average loss from 1000 ft. of pipe before testing is about 200 drops per min., equivalent to about 90 gal. per mi. per day. Leaks from joints in Washington[105] were generally due to insufficient lead, which allows a crevice to open under slightest deflection. By going over the joints, leakage may be reduced so there is no visible escape of water. Reliance cannot be placed on behavior of a pressure gage in judging tightness of pipe.

For leakage permissible in new work, general practice appears to justify 60 to 250 gal. per day per mi. per in. of diam. Loweth[101] proposes 60 to 80 gal. per in. mi. Gregory, in improvements to Columbus water supply, specified as in Table 120.[102] Leakage specified at Akron[120] was 200 gal. per in. mi. per day; actual tests were near 70. Burns and McDonnell in many systems in Middle West specified 80.[100]

* Reader is referred to, "Reduction of Water Consumption by Means of Pitometer Survey and Constant Inspection," by Andrews, *J. A. W. W. A.*, Vol. 6, 1919, p. 355; "Control of Water Waste by House to House Inspection," by Smith, *J. N. E. W. W. A.*, Vol. 35, 1921, p. 322, and to questionnaire in *Munic. J.*, June 15 and 22, 1918.

† For methods in New York City, see *Munic. Eng. J.*, Paper 137, 1923, p. 23.

‡ See also p. 467.

§ See [illegible]

Table 119. Total Underground Leakage in Cast-iron Water-pipe Systems in Operation

City	Miles of cast-iron pipe	Size, in.	Leakage, gal. per day per mi.	Leakage, gal. per day per in. of diam. per mi.†	Date	Remarks
Hoboken, N. J.						
High Service	22.37	4–16	1,285 *a*	160*	1883	Metered in and out.
Same	22.37	4–16	12,600 *a*	1,580*	1888	Metered in and out.
Englewood, N. J.	4.78	4–8	1,355 *a*	226*	1888	Metered in and out.
Fall River, Mass.	Whole	System	10,000 *b*	————	1904	96 % metered.
Fall River, Mass.	Whole	System	—————	600* *c*	1900	
New York, N. Y.	Whole	System	142,000 *d*	————	1900	
New York, N. Y.	Whole	System	————.	13,100* *c*	1900	
Boston, Mass.	Whole	System	14,187 *d*	————	1900	
Newton, Mass.	Whole	System	3,832 *e*	————	1897	
Brockton, Mass.	Whole	System	6,200 *b*	————	1904	90 % metered.
Ware, Mass.	Whole	System	11,200 *b*	————	1904	100 % metered.
Worcester, Mass.	Whole	System	20,800 *b*	————	1904	94.5 % metered.
Wellesley, Mass.	Whole	System	3,450 *b*	————	1904	100 % metered.
Yonkers, N. Y.	Whole	System	23,340 *b*	————	1904	100 % metered.
Woonsocket, R. I.	Whole	System	4,370 *b*	————	1904	86.7 % metered.
Milton, Mass.	Whole	System	3,110–3,680 *b*	————	1904	Metered in and out.
Belmont, Mass.	Whole	System	2,130–4,780 *b*	————	1904	Metered in and out.
Melrose, Mass.	1.33	4–14	43,770 *b*	6,100	1904	District tests.
Chicago, Ill.	25.8	————	202,500 *f*	————	1910	Streets tested before paving.
Milwaukee, Wis.	8.72	4–24	75,000 *g*	7,500*	1911	District tests.
Milwaukee, Wis.	13.3	6–20	48,800 *g*	4,880*	1911	District tests.
Washington, D. C.	83.0	3–20	76,500 *h*	10,600	1911	District tests.
Providence, R. I.	1.0	————	39,654 *i*	————	———	Test in mill yard pipe 36 years old.
Providence, R. I.	1.0		9,263 *i*	————	———	Same after repairing.
Providence, R. I.	5.57	12–24	2,478 *i*	————	1900	High pressure 114 lb., fire service 3 years old by meter.
Brooklyn, N. Y.	15.39	————	261,500 *j*	————	1910	Summary of district tests for years.
New York, N. Y.	28.1	12	7,380 *j*	615	1909	Bypass at pumping stations.
Akron, Ohio	2.0	6	10,000 *k*	1,667	1911	Street tests.
Grandview Heights	5.5	6–12	2.3 *k*	0.31	1911	Metered in and out.
Corpus Christi, Tex.	14.1	18	1,800	89	1915	Specifications required 144.[100]
Glencoe, Ill.	15.5	4–10	—————	800	1896	Meter checked midnight readings.[101]
Milton, Mass.	32	————	3,372	————	1901	101
	————	————	—————	.720–960	———	Conclusion from tests of 10-in. lines and smaller.[102]
Kincaid, Ill.	5	4–10	2,370	470	1914	102
Gary, Ind.	20	6–30	—————	2,600	1909	102
Hartford, Conn.	3.2	8–20	2,867	0.64‡		

* Approximations from data given. *a*, Brush; *b*, Brackett; *c*, Freeman; *d*, Croes; *e*, Kuichling; *f*, Phillips; *g*, Palmer; *h*, Garland and McFarland; *i*, I. S. Wood; *j*, New York Report; *k*, Bradbury. In Washington, D. C., for years 1908, 1909, 1910, and 1911, leakage from pipe joints was, respectively, 24, 20, 16, and 37 per cent. of total leakage, average 24 per cent.

† Leakage per in. of diam. per mile $\times\ y$ = Leakage per lin ft. of joint (time intervals the same) $y = 0.0060$ for 4 in.- and 0.0075 for 20-in. pipe (inside diam. of bell used).

‡ Per lin. ft. of joint, gals. per 24 hr.

Table 120. Allowable Limits of Leakage from New Cast-iron Water-pipe Lines, Columbus, O.[102]

Pressure 110 lbs. per sq. in. — Size, inches	Leakage, allowable gals. per hr. per linear ft.	Equivalent in gals. per 24 hrs. per mi.	Gals. per 24 hrs. per in.-mi.*
20	0.08	10,138	507
24	0.10	12,672	528
36	0.15	19,008	528

* Leakage figures reduced to unit lengths of mains are clai[illegible] Cast-iron Pipe Publicity Bureau, to be unfair; a great quantity of leakage is through the [illegible]

Leakage Records.* It is customary to allow 3 gal. per day per ft. (2.67 gal. per lin. ft. of joint per 24 hr.) for 48-in. cast-iron mains in Brooklyn;† tests on 48-in. standard cast-iron pipe resulted in a leakage of 7900 gal. per mi. per day (1.12 gal. per lin. ft. of joint per 24 hr.) under 128- to 163-lb. test pressure. A line in Staten Island under pressures from 47 to 116 lb. showed a leakage of 6700 gal. per mi. per day (1.27 gal. per lin. ft. of joint per 24 hr.). In removing 50 mi. of cast-iron mains in Brooklyn, 90 per cent. being 6 in., and the rest 8 to 12 in., only a few dripping joints were found, most of them in a block where the pipe was laid without calking. A 48-in. main sagged 26 in. in six lengths without noticeable leakage. On a force main at Ridgewood laid with solid lead joints, three times the leakage of ordinary joints has occurred.

A 48-in. steel main in Philadelphia, after being tested and recalked, had leakage of 7000 to 10,000 gal. per mi. per 24 hr. (based on field joints every 30 ft., leakage per lin. ft. of joint per 24 hr. = 3.1 to 4.5 gal.) under 160-lb. pressure. A certain 72-in. steel main gave following test results: Length of section, mi., 3.57, 0.19, 3.43, 0.26, 3.35; leakage per mi., cu. ft. per sec., 0.024, 0.039, 0.034, 0.370, 0.098, respectively. Pressure was 0 to 55 lb.; 10.8 mi. averaged 0.059 cu. ft. per sec. per mi. (Based on field joints every 30 ft., leakage per lin. ft. of joint per 24 hr. = 0.000012 mg.) General experience with steel mains is that they tend to grow tighter with age, provided corrosion does not pierce plate. See also p. 356.

Leakage from Services. Investigations in Minneapolis[106] in 1918 showed that 37 per cent. of leaks were from defective wiped joints; workmen have not the old skill. Couplings are preferred for longer life and less leakage. Western New York Water Co. allows lead-flanged union connections only.[12] Custom in some cities is to lay service pipe to vacant-lot line before paving. Of services so laid in Chicago in 1911, 55 per cent. were for future, and were a prolific source of leakage.[107] Tests by Wolfe[108] in stockyards, Chicago, where practically all services are cast-iron pipe, indicated but 2.3 per cent. of water unaccounted for. In tests in other parts of city, main leakage was but 7 per cent., while that in services was 67 per cent., and in valves and hydrants 26 per cent. Only one-tenth of leakage comes from mains; yet they are used as basis for the leakage unit.§

ELECTROLYSIS OF WATER PIPES

Effect. Electrolysis injures or destroys, sometimes rapidly, cast-iron, wrought-iron, steel, and lead mains and service connections, and lead sheaths of cables.‡ American Committee on Electrolysis representing national engineering societies and interested associations is formulating its third report. Reports dated 1916 and 1921 may be consulted in libraries, and the latter purchased from A. I. E. E., New York.

Kinds. Stray-current electrolysis is due to return or stray currents from electric railroads, telephone, or other electric-service conductors, which traverse the pipes as part of the circuit, and at points of departure carry away particles of the metal; also, but less extensively, due to local formation of a

* See also Bradbury, *Ohio Eng. Soc.*, Jan. 1912.

† Joints are computed on inside diam. of pipe.

‡ Reinforced concrete pipe is also affected; see Technological Paper No. 52, 1924, Bureau of Standards.

§ See

battery, as when an electrolyte is present in surrounding soil, and steel with mill scale supplies the other two elements. Earth currents also cause electrolysis, especially in damp ground and along water courses; this last is termed *soil corrosion* (see p. 387).

Cause. When electric current passes through a metal conductor, it causes no chemical change, but when it passes through a conductive medium which is decomposable by the current (termed an electrolyte), decomposition of the metal occurs at points where the current leaves. Pure water becomes an electrolyte only on addition of certain salts, which are in most soils.

Conditions. Electric corrosion occurs only when and where current leaves the pipe or cable sheath; this may be (*a*) at insulating or other high-resistance joints where current passes from one pipe length to the next through the earth, or (*b*) where current leaves the pipe line permanently. Removal of metal starts pitting, which may develop into holes.

Surveys. To determine geographical extent and intensity of conditions causing electrolysis of pipe systems, electric surveys are made, measuring differences of potential at various points between pipes and electric railway or other near-by conductor of current; direction of current with respect to pipes is also determined. Since numerous obscure elements enter most electrolysis problems, surveys are satisfactorily made only by an experienced specialist. Rate and extent of damage vary greatly and are influenced by many local conditions. For example, under a large portion of Newark, N. J., there is light, dry sand very unfavorable to electrolysis; Cincinnati, Ohio, reports no damage, but its electric railways use the double-trolley system; in Altoona, Pa., and many other places trouble is attributed to insufficiently bonded rails of electric roads.

Remedies.* Most methods have proved of limited application or useless. (1) Paints, insulating papers, and textiles do more harm than good. Cement coatings are porous in spots and afford no permanent protection. (2) Cathodic devices for maintaining the pipes negative to earth are deemed inexpedient or worthless. (3) Location of pipes as far as practicable from street railway tracks. (4) Insulating joints, if sufficiently frequent, sometimes protect the pipe. Frequency depends on potential gradient. Cement, leadite, and metalium make high-resistance joints, but the value of leadite decreases with age. Such joints are also employed to prevent interchange of current between two piping systems. Insulated joints do harm where there are currents across the pipe between other subsurface pipes, as the intermediate pipe is affected at point of leaving far more seriously than if it were part of a continuous conductor. East Bay Water Co., California, insulates services wherever possible, both at main and on house side of meter. (5) Drainage consists in connecting the affected structure to the railway return circuit by insulated conductors in such a manner that the current leaves the structure

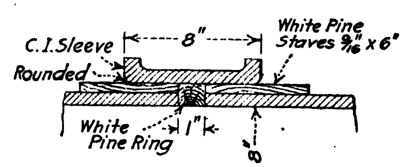


FIG. 184.—Inserted coupling, designed by McKenna.[112]

* In some communities, notably at Council Bluffs and Omaha, [illegible]rative committees have been financed by utility companies.

through these connections instead of flowing to earth. Drainage may improve conditions in one part of pipe at expense of another part. (6) Three-wire system of distribution theoretically holds promise, but in practice the desired electrolytic equilibrium has not been attained, and serious corrosion has resulted; used at Winnipeg and elsewhere.[111]

Electrolysis Safeguards on Steel Conduits for Raw Water, Cleveland. The twin lines lie in a neutral area. Where cast-iron specials or valves occur, a copper bond connects steel to steel; where more than one piece makes up the special, a branch copper line connects each part with the main bond, so that there will be no occasion for a stray current to jump a lead joint. Gaging boxes have been installed at strategic points, and copper lines installed for determining both the direction and the amount of electrical flow along the pipe as well as through the soil away from the pipe. These gaging manholes will eliminate excavations for electrolysis measurements.

EQUIPMENT AND APPURTENANCES

Drinking Fountains. Committee* of A. W. W. A. reported on sanitary aspects as follows (1924):

1. All types of drinking fountains with vertical jets are to be condemned.
2. Most types of drinking fountains with slanting jets are to be condemned.

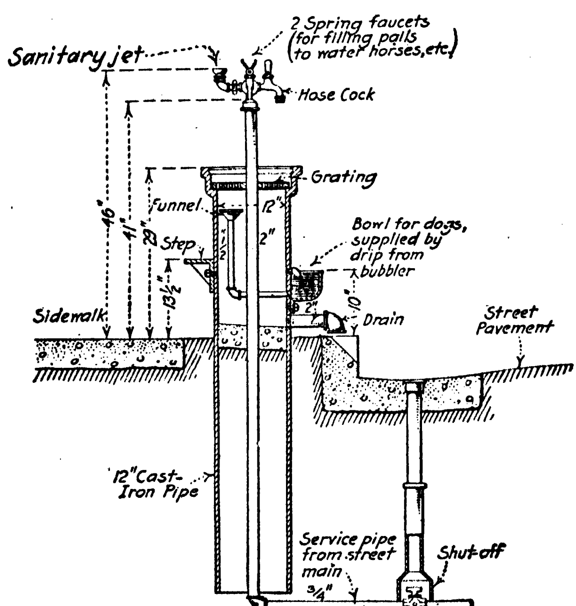


FIG. 185.—Public watering station.
(Designed by Frank E. Merrill, Water Commissioner, Somerville Mass.[114])

3. To be sanitary, drinking fountains should conform to following specifications:
a. Jets shall be slanting.

* The committee re[illegible] th[illegible] great variety of fountains put out by manufacturers are not yet of approved t[illegible]

b. Orifices of jets shall be protected in such a manner that they cannot be touched by fingers or lips, or be contaminated by droppings from mouth, or by splashings from basins beneath orifices.
c. Guards of orifices shall be so made that infectious material from mouth cannot be deposited upon them.
d. All fountains shall be so designed that their proper use is self-evident.

Section 61 of the Railway Sanitary Code of U. S. Public Health Service specifies:

If drinking fountains of bubbling type are provided in any railway station, they shall be so made that drinking is from a free jet projected at an angle to the vertical and not from a jet that is projected vertically or that flows through a filled cup or bowl.

SERVICE PIPES*

Sizes. The standard size of service connecting house with mains is ¾ in., although 1 and even 1¼ in. have been used where rapid internal corrosion was feared. Ordinarily, 1-in. pipe can go twice as long between cleanings as ¾ in., as it has twice the area. New England W. W. Committee,[115] recommended large service sizes where metered; use on unmetered supplies leads to waste, particularly through garden hose. Size of tap is not yet standardized; ⅝ in. is smallest size tap now used.

Connections to Mains. Services are joined to mains through corporation cocks: small valves operated by turning 90 deg. with a wrench, with a small aperture opposite the main through which the service pipe may be drained when cock is closed. Sometimes cock is replaced by a gate valve, to provide control from surface. Ordinarily, service is controlled from a shut-off valve near the curb protected by a curb box. Service curb boxes† afford means of controlling the services from curb to house. Control of whole service pipe is now Boston practice; a second stopcock with box is installed at the main.

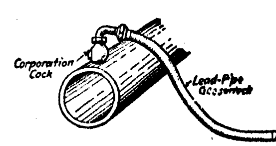


FIG. 186.

Gooseneck (Fig. 186) connects corporation cock to service, and furnishes flexibility to maintain integrity of connection if main settles. In favorable soils, particularly in New England, gooseneck is omitted and rigid connection made, enabling service to be cleaned readily from cellar to main by rodding. Due to breakages of lead goosenecks Hartford substituted two elbows, and secured sufficient flexibility.[126]

Materials for Services. Plain iron or steel, galvanized iron or steel, lead, brass, lead-lined, and cement-lined pipes have been used. Corrosion, freezing, and electrolysis affect life of service pipe, which varies from 16 to 50 years, dependent on conditions and materials. Researches by Speller‡ and others (see pp. 587, 810) on deactivation of water have increased our knowledge of corrosive agencies. Many services depreciate from external corrosion;§ surrounding with concrete has been tried as a remedy. Galvanized pipe is

* Committee 10, A. W. W. A., is to report on service standards. See Manual of American Water Works Practice, 1925, p. 385.
† Service boxes are made by H. W. Clark Co., Mattoon, Ill.; S. E. T. Valve & Hydrant Co., New York; W. G. Classon, Leominster, Mass.; and others.
‡ See "Corrosion" (McGraw-Hill Book Company, Inc., 1926).
§ See Donaldson in *J. A. W. W. A.*, Vol. 11, 1924, p. 649.

most common material; protective measures are relatively simple and inexpensive. Lead appears to give longest life; its use is restricted to waters where plumbism does not threaten. See questionnaire on life, in *Public Works*, 1922, pp. 345 and 352–355. Cement-lined pipe has been used in New England for 40 years (see p. 374). It can be bent to a 5-ft. radius; recent improvement in lining should lead to wider adoption, as it gives a service free from corrosion or plumbism (see Newsom, in *J. A. W. W. A.*, Vol. 13, 1925, p. 145).

Lead services are extensively used. The *advantages* are: (*a*) long life; (*b*) freedom from rusty waters. The *disadvantages* are: (*a*) Lead has been found in many instances to constitute a grave menace to health, particularly with soft waters, colored, swampy waters, and waters containing carbonic acid.[127] (*b*) The cost of installing it is more than for galvanized pipe.

Galvanized service pipes are most extensively used. *Advantages* are: ease of installation, because of threaded joints and fittings; (*b*) cheapest piping obtainable. *Disadvantages* are: (*a*) The galvanizing does not permanently protect against corrosion. (*b*) All iron or steel pipe, irrespective of whether it is galvanized, corrodes; results are red dirty water, short life with consequent replacement, leakage.

Cast-iron Services. Generally used only where size is 2 in. or over and preferably for 4-, 6-, or 8-in. connections. Very limited use for ordinary domestic purposes.

Tin-lined Lead Pipe. It has been shown that the tin lining does not prevent the taking into solution of lead.

Cement-lined Services. *Advantages* are: (*a*) no rust, (*b*) no harmful corrosive product. *Disadvantages* are: (*a*) making satisfactory joints; (*b*) cost of installing; (*c*) difficulty of producing satisfactory lining.

Brass* and Copper Services. *Advantages* are: (*a*) possibility of eliminating the lead gooseneck entirely by using soft pipe that may be bent to the main; (*b*) no rusty water; (*c*) minimum of corrosion (no rust clogging); (*d*) easily installed; (*e*) no harmful corrosion products and no poisoning; (*f*) no replacing necessitating the tearing up of expensive pavement as with corrodible piping; (*g*) ultimate saving effected where replacements are necessary; (*h*) no inconvenience due to shutting off water for replacement. *Disadvantages* are: costs to install about 10 to 12 per cent. more than galvanized piping and about 1 to 3 per cent. more than lead services.

Plain wrought-steel pipe, termed "wrought pipe,"† often used by unscrupulous building contractors to reduce first cost, has a short life; Wolfe considers it responsible for large underground leakages.[108]

Grounding Secondary Electric Circuits. McCullom and Peters[116] claim no danger of electrolysis from a.c. circuits; they recommend that water pipes only be used for grounding; there is danger of explosions from grounding on gas pipes.

Standards. Committee of N. E. W. W. A., reported in 1916, and A. W. W. A. has a committee formulating standards (1926).

Laying. Services can be laid in trenches or can be jacked through an earth section by a "pipe pusher"‡ which will handle ¾- to 2-in. pipe.

* Adopted by Hackensack Water Co.
† See p. 349.
‡ [illegible]de by Water Works Equipment Co.

Table 121. Water-pipe Sizes* for Plumbing Fixtures, Branches, Risers, and Mains

(*A*) Supply Sizes for Fixtures and Maximum Flow in Gal. per Min.

	Number of fixtures									
	1	2	4	8	12	16	24	32	40	
Water closets:										
Gal. per min.	8	16	24	48	60	80	96	128	150	Tanks
Pipe size	½	¾	1	1¼	1½	1½	2	2	2	
Gal. per min.	30	50	80	120	140	160	200	250	300	Flush valves
Pipe size	1	1¼	1½	2	2	2	2½	2½	2½	
Urinals:										
Gal. per min.	6	12	20	32	42	56	72	90	120	Tanks
Pipe size	½	¾	1	1¼	1¼	1¼	1½	2	2	
Gal. per min.	25	37	45	75	85	100	125	150	175	Flush valves
Pipe size	1	1¼	1¼	1½	1½	2	2	2	2	
Lavatories and wash sinks—based upon each faucet:										
Gal. per min.	4	8	12	24	30	40	48	64	75	....
Pipe size	½	½	¾	1	1	1¼	1¼	1½	1½	....
Bath tubs:										
Gal. per min.	15	30	40	80	96	112	144	192	240	....
Pipe size	¾	1	1¼	1½	2	2	2	2½	2½	....
Shower baths:										
Gal. per min.	8	16	32	64	96	128	192	256	320	8 in. rain
Pipe size	½	¾	1¼	1½	2	2	2½	2½	3	head
Acid and slop sinks, manufacturing, kitchen, and laundry:										
Gal. per min.	15	25	40	64	84	96	120	150	200	Per bibb
Pipe size	¾	1	1¼	1½	1½	2	2	2	2½	Per bibb

Note.—The above sizes are based upon a pressure drop of 30 lb. per 100 ft.
In estimating risers and mains, the number of gal. for water closets and urinals, where flush valves are used, are to be as given for tanks.
The hot-water faucets are to be disregarded when estimating risers and mains.

(*B*) Sizes of Water Supply Branches

Fixture	Number of fixtures					
	1	2	4	8	12	16
Water closets:						
With tanks	½	¾	1	1¼	1½	1½
With flush valves	1	1¼	1½	1½	2	2
Urinals:						
With tanks	½	¾	1	1¼	1¼	1¼
With flush valves	1	1¼	1¼	1½	1½	2
Lavatories and washing sinks:						
Per bibb	½	½	¾	1	1	1¼
Bath tubs	¾	1	1¼	1½	2	2
8 in. shower	½	¾	1¼	1½	1½	2
Slop and acid, manufacturing and laundry sinks, per bibb	¾	1	1¼	1½	1½	2

* W. S. Timmis.

Head Lost. Many investigations have been made. See Foss' formula, *J. A. E. S.*, Vol. 13, 1894, p. 295, and "Flow of Fluids through Commercial Pipe Lines," by Wilson, McAdams, and Seltzer, *E. N. R.*, Oct. 26, 1922, p. 690.

Bleistein's Table. This was calculated from test data, and indicates the pressure difference between the street main and water flowing in a lead service

pipe 30 ft. distant from the main, with various sizes of service pipes. The table is helpful in deciding on the effect of sizes. For instance, the output with 2-in. pipe with a 1-in. corporation cock is cut down nearly to that of a 1½-in. pipe with a 1½-in. cock. The additional loss, due to strainer, is also shown.

Table 122. Loss of Head in Corporation Cocks and Service Pipes. Bleistein[117]
(First line gives diameter of lead service pipe; second line, cock size)

	2 inch							1½ inch					1 inch			¾ inch		⅝ inch
Flow, gallons per minute	2 inch	2 inch†	1½ inch	1½ inch†	1 inch	¾ inch	⅝ inch	1½ inch	1½ inch†	1 inch	¾ inch	⅝ inch	1 inch	¾ inch	⅝ inch	¾ inch	⅝ inch	⅝ inch
5	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	2	2	3
10	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	5	6	10
15	...	...	...	...	...	...	...	...	...	...	...	...	3	4	5	11	12	23
20	...	...	...	...	...	...	5	...	...	...	...	5	5	6	9	18	21	28
25	...	...	...	...	...	3	7	...	...	...	4	8	8	10	14	28	32	54
30	...	...	...	...	...	5	11	...	...	2	5	12	11	14	20	39	46	...
40	...	...	...	...	3	8	19	2	3	4	10	21	18	24	35	69	80	...
50	...	...	...	...	4	13	30	4	5	7	15	32	28	37	54	...	...	...
60	...	...	...	3	6	18	43	5	7	9	22	47	39	51	76	...	...	...
70	...	...	...	4	8	25	63	7	9	13	30	68	52	69	...	...	...	...
80	...	...	...	5	10	33	...	9	11	17	40	...	68	...	...	...	...	...
90	...	3	3	6	13	42	...	12	14	21	52	...	...	...	...	...	...	...
100	3	4	4	7	16	51	...	14	17	26	...	...	...	...	...	...	...	...
125	5	6	7	12	25	77	...	22	27	40	...	...	...	...	...	...	...	...
150	8	9	10	17	36	...	...	30	38	57	...	...	...	...	...	...	...	...
175	10	12	13	23	49	...	...	41	51	77	...	...	...	...	...	...	...	...
200	13	16	17	29	62	...	...	52	65	...	...	...	...	...	...	...	...	...
225	17	20	21	37	...	...	...	65	...	...	...	...	...	...	...	...	...	...
250	20	24	25	45	...	...	...	...	...	...	...	...	...	...	...	...	...	...
275	24	29	30	55	...	...	...	...	...	...	...	...	...	...	...	...	...	...
300	29	35	36	64	...	...	...	...	...	...	...	...	...	...	...	...	...	...

† With strainer.

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CHAPTER 20

VALVES, SLUICE GATES, AND HYDRANTS

GATE VALVES

Standard Gate Valves. Standard specifications for valves, adopted by the A. W. W. A., July, 1913, are universally accepted by makers* and users; they were established as railway practice in 1922 by Am. Ry. Eng. Assn.† On small municipal systems, it is common practice to omit detailed specifications, but to stipulate that valves shall conform to the standard. Certain local conditions must always be met (see Ordering Valves). Only on large works and for special conditions are valves designed by the engineer. For New York City standards, see *E. N.*, May 27, 1915, p. 1016. The designing engineer, particularly on pumping-station or filter-plant work, will find catalogs of valve companies indispensable for working dimensions.

Ordering Valves. Laying lengths of valves of the different makers vary considerably; keep this in mind when ordering for a restricted space. When ordering valves or taking bids specify: (*a*) Ends to be double-hub, double-spigot, double-flange, a combination of these, or universal.‡ (Some makers consider a double-flange valve in the larger sizes as special and charge a higher price.) (*b*) If flanged, is drilling standard? If not, give particulars. (*c*) By-pass omitted? (*d*) Diameter. (*e*) Purpose: water, steam, gas, etc. (*f*) Materials: all iron, all bronze, bronze mounted. (*g*) Is valve axis to be vertical, horizontal, or inclined? (*h*) Rising or stationary spindle? (*i*) Direction of operation clockwise or reverse? (*j*) If hand-operated, hand wheel or key to be used? (*k*) If hydraulically operated, state operating pressure. (*l*) If electrically operated, state kind, voltage, frequency, and phase of current. (*m*) Test pressure. (*n*) Painting. (*o*) Double disk, single disk, or wedge. (*p*) If geared, spur, or beveled? Valves with a quarter bend on one end; termed "angle valves," are a convenience in a restricted space.

Solid-gate vs. Double-disk Valve. Former is Boston standard; latter may be termed the commercial type. Boston[1] continued its standard for the following reasons: 67 years of experience had developed no convincing reason for discarding it; no inherent weakness or objectionable characteristics have been observed; it has met requirements satisfactorily. When a double-disk valve is used for throttling, vibrations injure it. Ruggedness of solid gate withstands throttling stresses; alterations can prevent chattering. Makers of solid-gate valves include Jenkin Bros., and Kennedy Valve Co.

* Standard valves are made by many firms, including Chapman Valve Mfg. Co., Indian Orchard, Mass.; Coffin Valve Co., Boston, Mass.; Coldwell-Wilcox Co., Newburgh, N. Y.; Crane Co., Chicago; Flowers-Stephens Mfg. Co., Detroit, Mich.; Eddy Valve Co., Waterford, N. Y.; Kennedy Valve Mfg. Co., Elmira, N. Y.; Ludlow Valve Mfg. Co., Troy, N. Y.; Michigan Valve & Foundry Co., Detroit; Rensselaer Mfg. Co., Troy, N. Y.; A. P. Smith, East Orange, N. J.; R. D. Wood & Co., Philadelphia; Iowa Valve Co., Oskaloosa Ia.; Darling Valve & Mfg. Co., Williamsport, Pa.

† Specifications for both furnishing and installing all types of water valves issued by Assn. Factory Mutual Fire Ins. Cos., Boston, 1921.

‡ Whatever the kind of ends, make sure they will fit the pipes, specials, or other valves with which they are to be connected.

Valve stem is the most important part of a valve. It operates in both tension and compression. The greatest strain is in wedging the gate to the seat. Strength and non-corrodibility are essential. Brass and rolled bronze are generally used (see Chap. 36). B. W. S., New York, specified manganese bronze. See Conard, *J. N. E. W. W. A.*, Vol. 36, 1922, p. 35.

Circular Sluice Gates and Gate Valves. Areas of Openings. If the distance a gate or valve has been moved from its seat be known, by indicator or measurement on stem, Table 124 will give the area of opening; or if a gate or valve be opened at uniform speed and total time required to open wide be known, the area of opening at any instant, or any proportion of total time, will be given. Units, ft. and sq. ft. D = diam. and a = area of port of gate or throat of valve. H = distance from bottom of gate or plug to invert of pipe. A = area of opening (Fig. 187).

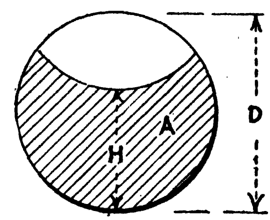


FIG. 187.

In using these tables to compute discharge through gate valves or circular sluice gates, it must be borne in mind that almost as soon as the valve or gate is started from its seat, there will be a space all around through which water can flow in increasing quantity as the valve or gate moves, before there is any waterway area of the form shown in Fig. 187. Likewise, the stem must make a certain number of turns, or it and the valve plug or gate must move a distance somewhat greater than seat facing on valve or gate before H begins to have a value.

Table 123. Valve vs. Pipe Sizes

Location	Pipe size, inches	Valve size, inches
Brooklyn	72	48
Jersey City	72	48
City Tunnel, Catskill Aqueduct	144	66
Brooklyn	48	36
Bayonne	48	36 and 30
Springfield, Mass	42	30
Coolgardie	30	20
Chicago[37]	42 to 48	36
	30 to 36	24
	20 to 24	16
Cleveland filters	60	48

* Lally has demonstrated the small hydraulic losses involved.[2]

Selecting Size of Valve. Funds and operating time can be wasted by selecting needlessly large valves. There is an increasing tendency to use valves of smaller sizes than the mains,* with increasers and reducers on all sizes above 16-in.[5] On 48-in. lines in Boston, 36-in. valves were used.[6] New York now uses but six sizes:

Pipe, inches	6	8	12 & 16	20, 24, 30	36 & 48	Above 48
Valve inches	6	8	12	20	36	48

For designs and specifications, see *E. N.*, May 27, 1915, p. 1016. Number of valves and repair parts necessarily kept in stock is much reduced. By using

smaller valves in larger pipes, shut-offs can be more quickly made and less room is occupied. Other examples of practice are given in Table 123.

Table 124. Areas of Openings of Circular Gate Valves and Sluice Gates

Time or H/D ratio	$\frac{A}{a}$	Valve Size 60 in.	48 in.	36 in.	24 in.	20 in.	16 in.	12 in.
		Actual areas, A, sq. ft.						
0.05	0.063	1.236	0.791	0.445	0.197	0.137	0.088	0.049
0.10	0.127	2.493	1.595	0.897	0.399	0.277	0.177	0.099
0.15	0.190	3.729	2.387	1.343	0.596	0.414	0.265	0.149
0.20	0.253	4.986	3.191	1.795	0.798	0.554	0.354	0.199
0.25	0.315	6.203	3.970	2.233	0.992	0.689	0.441	0.248
0.30	0.376	7.572	4.737	2.664	1.184	0.822	0.526	0.296
0.35	0.436	8.578	5.491	3.088	1.372	0.953	0.610	0.343
0.40	0.495	9.756	6.245	3.513	1.561	1.084	0.693	0.390
0.45	0.553	10.875	6.961	3.915	1.740	1.208	0.773	0.435
0.50	0.610	11.974	7.665	4.311	1.916	1.330	0.851	0.479
0.55	0.663	13.034	8.343	4.693	2.086	1.448	0.927	0.521
0.60	0.715	14.074	9.009	5.068	2.252	1.564	1.001	0.563
0.65	0.765	15.036	9.625	5.414	2.406	1.671	1.069	0.601
0.70	0.811	15.959	10 216	5.746	2.554	1.773	1.135	0.638
0.75	0.855	16.842	10.781	6.064	2.695	1.871	1.197	0.673
0.80	0.895	17.588	11.259	6.333	2.814	1.954	1.251	0.703
0.85	0.932	18.334	11.736	6.602	2.934	2.037	1.304	0.733
0.90	0.962	18.923	12.113	6.814	3.028	2.103	1.345	0.757
0.95	0.987	19.394	12.415	6.983	3.103	2.155	1.379	0.776
1.00	1.000	19.634	12.566	7.068	3.141	2.181	1.396	0.785

Table 125. Areas of Openings of Circular Gate Valves, Sq. Ft.
(Function of Turns of Valve Stem)

Supposing a valve to be raised at a uniform rate by turning the stem at a uniform speed, this table shows the size of the gate opening at any time; also the number of revolutions required to produce the area, based on the number of turns necessary to effect a complete opening.

Time or H/D	60-in. pipe: No. of turns 124 (*b*)	496 (*a*) (*k*)	620 (*c*) (*j*)	Area, sq. ft.	48-in. pipe: No. of turns 100 (*l*) (*d*) (*b*) (*g*)	275 (*a*)	300 (*c*)	400 (*e*) (*h*)	436 (*j*)	500 (*i*) (*j*) (*k*)	Area sq. ft.
0.05	6	25	31	1.236	5	14	15	20	22	25	0.791
0.10	12	50	62	2.493	10	28	30	40	44	50	1.595
0.15	18	75	93	3.729	15	42	45	60	66	75	2.387
0.20	25	99	124	4.986	20	55	60	80	87	100	3.191
0.25	31	124	155	6.203	25	69	75	100	109	125	3.970
0.30	37	149	186	7.572	30	83	90	120	131	150	4.737
0.35	43	174	217	8.578	35	97	105	140	153	175	5.491
0.40	50	198	248	9.756	40	110	120	160	174	200	6.245
0.45	56	223	279	11.128	45	124	135	180	196	225	6.961
0.50	62	248	310	11.974	50	138	15C	200	218	250	7.665
0.55	68	273	341	13.034	55	152	165	220	240	275	8.343
0.60	74	298	372	14.074	60	165	180	240	262	300	9.009
0.65	80	323	403	15.036	65	179	195	260	284	325	9.625
0.70	87	347	434	15.959	70	193	210	280	305	350	10.216
0.75	93	372	465	16.842	75	206	225	300	327	375	10.781
0.80	99	397	496	17.588	80	220	240	320	349	400	11.259
0.85	105	422	527	18.334	85	234	255	340	371	425	11.736
0.90	112	446	558	18.923	90	248	270	360	392	450	12.113
0.95	118	471	589	19.394	95	261	285	380	414	475	12.415
1.00	124	496	620	19.634	100	275	300	400	436	500	12.566

Table 125. Areas of Openings of Circular Gate Valves.—(*Continued*)

Time or H/D	36-in. pipe								24-in. pipe				
	Number of turns							Area, sq. ft.	Number of turns				Area, sq. ft.
	76 (g) (l) (b) (d)	209 (a)	228 (c)	240 (h)	250 (k) (e)	274 (i) (f)	304 (j)		75 (l) (b) (g) (m) (a)	102 (k)	152 (h) (e) (j)	225 (i) (f)	
0.05	4	10	11	12	13	14	15	0.445	4	5	8	11	0.197
0.10	8	21	23	24	25	27	30	0.897	8	10	15	22	0.399
0.15	12	31	34	36	38	41	45	1.343	12	15	23	33	0.596
0.20	15	42	46	48	50	55	61	1.795	15	20	30	45	0.798
0.25	19	52	57	60	63	69	76	2.233	19	25	38	56	0.992
0.30	23	63	68	72	75	82	91	2.664	23	31	46	67	1.184
0.35	27	73	80	84	88	96	106	3.088	27	36	53	78	1.372
0.40	30	84	91	96	100	110	122	3.513	30	41	61	90	1.561
0.45	34	94	103	108	113	124	137	3.915	34	46	68	101	1.740
0.50	38	104	114	120	125	137	152	4.311	38	51	76	112	1.916
0.55	42	115	125	132	138	151	167	4.693	41	56	84	124	2.086
0.60	46	125	137	144	150	164	182	5.068	45	61	91	135	2.252
0.65	50	136	148	156	163	178	198	5.414	49	66	99	146	2.406
0.70	53	146	160	168	175	192	213	5.746	53	71	106	157	2.554
0.75	57	157	171	180	188	206	228	6.064	56	76	114	169	2.695
0.80	61	167	182	192	200	219	243	6.333	60	82	122	180	2.814
0.85	65	178	194	204	213	233	258	6.602	64	87	129	191	2.934
0.90	68	188	205	216	225	247	274	6.814	68	92	137	202	3.028
0.95	72	199	217	228	238	261	289	6.983	72	97	144	214	3.103
1.00	76	209	228	240	250	274	304	7.068	75	102	152	225	3.141

Table 125. Areas of Openings of Circular Gate Valves.—(*Concluded*)

Time or H/D	20-in. pipe				16-in. pipe					12-in. pipe		
	Number of turns			Area, sq. ft.	Number of turns				Area, sq. ft.	No. of turns		Area, sq. ft.
	64 (g) (l) (m) (b) (d)	128 (e) (h)	192 (f) (i)		52 (g) (b) (l) (m) (d)	102 (e)	153 (h) (i)	159 (f)		40 (a) (f) (d) (l) (m)	78 (h)	
0.05	3	6	10	0.137	3	5	8	8	0.088	2	4	0.049
0.10	6	13	19	0.277	5	10	15	16	0.177	4	8	0.099
0.15	9	19	29	0.414	8	15	23	24	0.265	6	12	0.149
0.20	13	26	38	0.554	10	20	31	32	0.354	8	16	0.199
0.25	16	32	48	0.689	13	25	38	40	0.441	10	20	0.248
0.30	19	38	58	0.822	15	31	46	48	0.526	12	23	0.296
0.35	22	45	67	0.953	18	36	54	56	0.610	14	27	0.343
0.40	26	51	77	1.084	21	41	61	64	0.693	16	31	0.390
0.45	29	58	86	1.208	23	46	69	72	0.773	18	35	0.435
0.50	32	64	96	1.330	26	51	77	80	0.851	20	39	0.479
0.55	35	70	106	1.448	29	56	84	88	0.927	22	43	0.521
0.60	38	77	115	1.564	31	61	92	95	1.001	24	47	0.563
0.65	41	83	125	1.671	34	66	100	104	1.069	26	51	0.601
0.70	45	90	134	1.773	36	71	107	111	1.135	28	55	0.638
0.75	48	96	144	1.871	39	76	115	120	1.197	30	59	0.673
0.80	51	102	154	1.954	42	82	122	127	1.251	32	62	0.703
0.85	54	109	163	2.037	44	87	130	135	1.304	34	66	0.733
0.90	58	115	173	2.103	47	92	138	143	1.345	36	70	0.757
0.95	61	122	183	2.155	49	97	146	151	1.379	38	74	0.776
1.00	64	128	192	2.181	52	102	153	159	1.396	40	78	0.785

Ref.	Valve	Pressure	Gearing
a	Double gate	Light	None
b	Double gate	Light	Spur
c	Double gate	Light	Bevel
d	Double gate	Medium	None
e	Double gate	Medium	Spur
f	Double gate	Medium	Bevel
g	Double gate	Heavy	None
h	Double gate	Heavy	Spur
i	Double gate	Heavy	Bevel
j	Double gate, double stem	Heavy	Spur
k	Double gate, double stem	Heavy	Bevel
l	Double gate, outside screw & yoke	Heavy	None
m	Double gate	Extra heavy	None

Valve data are from catalog of Rensselaer Mfg. Co., Troy, N. Y.

Stresses in Valve Disks.[4] The following formulas apply to any segment of a sphere, being especially applicable to gate-valve disks of that form: Let r = radius of segment, in in.; v = versed sine or middle ordinate, in in.; p = hydrostatic pressure, lb. per sq. in., normal to the surface, acting radially toward the center. Stress at any point in the disk $= p(r^2 + v^2) \div 4v$. Stress in flange $= pr(r^2 - v^2) \div 4v$.

Location of valves in many systems are recorded only in memories of old employees. Maps should be prepared showing location, size, make, date set, repairs, and number of turns to open, for each valve, to facilitate emergency and other operations.[7] Gate valves on conduits should be placed at points convenient of access; their chief purpose is to restrict waste of water and damage to property when the pipe line is emptied or breaks occur (see p. 398). Many large conduits have no control valves. On distribution systems, valves should be so located that sections can be cut off with minimum inconvenience to consumers and fire risk; in high-value districts, not to exceed 600 ft.; in other districts, 900 ft.[36] For Pawtucket practice, see *E. N.*, May 6, 1915, p. 892.

Valve Boxes and Vaults. Valve boxes are cheap and easy to duplicate, but they do not protect the gears from dirt and ground waters as well as a vault. Vaults also allow access for lubrication, repairs, and replacements. On large lines it is generally necessary to place the valve horizontally and then the vault covers only the gearing, a box being used on the by-pass. For Salt Lake City standards, see *E. N.*, July 13, 1916, p. 66. Vaults for small valves, 2 by 3 ft. inside, cost $30 to $40 in Philadelphia. They are of precast reinforced concrete oval rings, the bottom course being split to straddle the pipe.* Concrete slab manhole containing cast-iron frame and cover is used on top.[9] In cold climates, shallow vaults should be filled with straw in winter.

MAINTENANCE AND OPERATION OF GATE VALVES

Care of Gate Valves and Hydrants. Neither valves nor hydrants of a public water system should be operated by others than employees of water and fire departments, or persons made definitely responsible. Valves and hydrants not frequently used should be inspected and operated periodically and all should be lubricated, painted, and repacked as found necessary. In localities having severe winters, freezing must be prevented. Packing with fresh horse manure is one effective way. If a hydrant is in wet ground, after each use, the drip should be plugged and water pumped out. A piece of lead on a string, guided by a loop of wire, pushed in through a hydrant nozzle, may be used to sound for ice in the barrel. If ice be found, it should be thawed with hot water. A few lumps of quicklime may be used instead, adding a little cold water, if necessary. A half-hour or so later the hydrant should be opened and the lime flushed out. Any water found standing in a hydrant barrel should be pumped out. Some superintendents put salt into troublesome hydrants during the winter to prevent freezing, others prefer a pint or a quart of wood alcohol, and some use a little crude glycerine. With a slow leak, the alcohol may rise to the top of the hydrant and the water beneath

* For Baltimore standards, see *E. N. R.*, Sept. 2, 1926, p. 374.

freeze solid. Furthermore, alcohol evaporates but glycerine does not. If an alcohol-and-water mixture freezes, the ice is mushy and does not prevent operation nor cause breakage by expansion. It is claimed salt has no effect on the brass or rubber. It has been found that a hydrant connection, thawed by forcing steam into the ground around it, will not freeze again the same winter. Valves will get closed accidentally when they should be open, or vice versa, usually through ignorance or carelessness; this must be guarded against. See *J. N. E. W. W. A.*, Vol. 28, 1914, p. 298.

Valve Insertion by Machine. Such machines are made by A. P. Smith Mfg. Co., and others. Two 20-in valves were inserted in a 20-in main under pressure at Springfield, Mass. See *E. C.*, Aug. 4, 1915, p. 98.

Lubrication. Operation twice a day of a 48-in. valve on a feeder line in New York City was greatly facilitated by installation of an oil cup. W. W. Brush[8] and C. E. Davis[9] complain that on large valves no provision is made for lubrication. Ludlow and some other geared valves are equipped with grease covers.

Packing for Stuffing Boxes. Italian hemp well soaked in a mixture of paraffin, vaseline, and a little lubricating graphite is durable, and does not corrode the valve parts. Organic fats should be avoided. Several makes of braided-wire and sectional packing are on the market; some of them are excellent. Lead wool has been used. Some superintendents repack all new valves on receipt. Lubrication of packing by tallow and graphite proved successful at Syracuse.

Gaskets. On penstocks, Hydraulic Power Comm.[11] found round gutta-percha gasket the most satisfactory. Placed in a groove in the flange, and held by rubber cement, it has withstood heads of 2100 ft. See p. 392.

Pressure in an Isolated Section. Ledoux[12] found that when a section under 105 lb. pressure was isolated by closing valves at either end and there was no flow in the section, a gage registered a pressure between the valves about 15 lb. less than anticipated from the head, due to the fact that the line was not absolutely tight.

Valve Chattering.* On Yakima project, U. S. Reclamation Service,[13] a standard Crane Co. 18-in. gate valve, installed in horizontal position, suffered breakage of tongue due to violent vibratory shocks caused by velocities of over 60 ft. per sec. On a similar valve on same project, care was taken in placing and operating; yet tremendous vibrations occurred when the valve opening was between $\frac{3}{8}$- and $\frac{3}{4}$- diam. An elbow below the valve was pierced with a 1-in. hole to admit air to valve chamber; this reduced the vibrations somewhat.

Valve throttling is often practiced when head must be killed, as on reservoir outlets. Solid-disk valves should be used in such situations. Ordinary valves should not be used, especially where tightness is desired when the valve is closed. Throttling a 36-in. valve for several hours each day for 9 years so as to create 15 or 20 ft. higher head (120 to 135 ft.) through another line had the following results: That part of the stem corresponding to one-fourth to one-sixth opening of the gate had its threads worn to a feather edge, so that it slipped through the nut; the seat of the valve was fluted to a maximum depth

* See Savage, "High-pressure Reservoir Outlets," U. S. Reclamation Service, 1923.

of $\frac{1}{16}$ in., presenting a rough sandy surface, as if particles of bronze had been removed along the lines of crystallization. The quantity of water delivered by throttling was probably 30 mgd.; velocity for full opening was 6.6 ft. per sec., approximately 25 ft. for one-fifth opening. The bronze used was probably so-called phosphor bronze, having a tensile strength of 30,000 to 40,000 lb. per sq. in.

Gate Valves. Power to Open. Information about power required to open large gate valves under high pressures is meager. Following data are reported to have been obtained with care: A pair of 48-in. gate valves with solid wedge-shaped plugs with bronze face rings; Babbitt metal seats in valve body; $3\frac{1}{2}$-in. bronze stems; 30-in. bronze-lined hydraulic cylinders, bolted to 31-in.-high distance or spacing castings, which in turn were bolted to the valve domes; pistons fitted with two cup-leather packings. Spindles passed through stuffing boxes on tops of domes and on bottoms of cylinders. Bronze indicator or tail rods from tops of pistons passed through stuffing boxes on tops of cylinders (top and bottom refer to valve when in upright position). Valves tested were installed in short 48-in. pipe lines, with axes of valves and cylinders inclined about 20° below a horizontal line. *Valve A:* Pressure per sq. in. on valve when closed, *i.e.*, pressure in 48-in. pipe on upstream side, 41 lb.; on downstream side about 2 lb.

Opening of valve, in.	0 (start)	3	6	12	15	18	24	Valve being opened
Pressure in cylinder, lb. per sq. in.	65	60	51	50	43.5	34.5	24	
Opening, in.	24+ (Started back)	24	18	15	9	3	Final	Valve being closed
Pressure in cylinder, lb. per sq. in.	12.5	18.5	23	31	40	50	115	

On repeated closing trials, final pressure ranged down to 76 lb. At 24-in opening, loud roaring was noticed, but no severe vibration.

Valve B: Upstream pressure, 12.6 lb.: downstream 2 ± lb.

	Being Opened					Being Closed		
Opening of valve, in.	0	3	6	9	9+	6	3	Final
Pressure in cylinders, lb. per sq. in.	31	17	15	13	11	16	17	20

Pressure in 30-in. Direct-connected Hydraulic Cylinder Required to Open 60-in. Solid-wedge Gate Valve under Pressure

Pressure to seat valve, lbs. per sq. in.	Pressure on back of valve plug, lbs. per sq. in.	Starting pressure in cylinder, lbs. per sq. in.	Coefficient of friction
35	52	15	0.22
35	49	15	0.216
50	50	17	0.24

Valve operation may be by hand, by electrical or water motors, by hydraulic cylinders. Hand operation is slow and expensive; 100 to 300 turns and about 45 min. time are needed to open a 48-in. valve. Hand operation in a plant involves some sort of floor stand equipped with a hand wheel or crank; in

the street, a key operating through a valve-stem box is used. Both methods are primitive compared to electrical or hydraulic operation. Valves must be closed slowly, and the necessity for this precaution increases with diameters. Otherwise sudden arresting of the momentum of the water* will create great pressure against the pipes in all directions, and throughout the length behind the gate. Unless operated by motor or hydraulic cylinder, valves usually operate much more slowly than is required on this account. It is sometimes desirable in small waterworks to have valves at remote reservoirs or tanks controllable from the superintendent's office or the pumping station. For long distances electric motors are the best means of operation; for shorter distances, hydraulic cylinders are also possible. For long distances electric transmission wires have numerous advantages over hydraulic pressure pipe. Valves thus operated from a distant point should have devices for showing, at the point of operation, the state of opening of each valve (see p. 437). Distance operation is important in filter plants, where groups of valves must be operated several times a day by one operator. Valves in a filter plant are under low head, so the lightest pressure valves are used.

Electrical Operation. Motor operation has proved satisfactory and is much more rapid than hand operation. Its most important application is to closing valves in case of pipe breaks, to restrict flooding. Electrical operation applied to the discharge valve of a centrifugal pump gives additional protection in case the check valve fails.† The electric motor is now adaptable to any and every possible use; it can be made watertight, and to work submerged or exposed to snow and ice.‡ Covered cable can be laid without additional protection in open trench or submerged. With large gates and valves in some situations the time required for hand work would be greater than emergencies would allow. In old-type installations, it was found necessary in some places to have a man at the switchboard when a valve is nearly seated, as a precaution against overseating and buckling of stem. This failure of motors to cut off when valves are seated was caused by the contacts connected with the automatic shut-offs or limit switches; if these contacts were set so that an opening valve cut off correctly, it was impracticable to set them so that a closing valve also shut off correctly. It was a common error to provide motors of insufficient power to start the valves after they had been closed a long time. Valves require more power to move them, generally, as they grow older.

Power-operating devices of Dean,§ Cory,‖ Kelty¶ and other** types can be attached to existing valves without a shutdown. Most valve companies, notably Chapman, Rensselaer, Kennedy, Ludlow, can equip new valves with electrical equipment. The claims for the Dean[14] equipment are: (1) single unit system; (2) attachable to existing valves without shutdown and with minimum effort; (3) positive in operation, with no reliance on momentum; (4) ample seating and unseating torque; (5) enclosed and waterproof. Payne Dean (New York) also makes an auto truck equipped for operating valves.

* For water hammer, see p. 781.
† At Detroit filters-check valves have been superseded by electrically-operated valves.
‡ On account of the excessive dampness, hydraulic valves are generally preferred in filter galleries.
§ Cutler-Hammer Co., Rochester.
‖ Chas. Cory & Son, Inc., New York.
¶ Coffin Valve Co.
** Crane Co. equips its valves with a device utilizing momentum to reduce the starting torque. General Electric Co. has recently developed a device.

For Buffalo experience in fitting Payne Dean apparatus to old valves, see *E. N. R.*, Feb. 2, 1922, p. 186. Payne Dean is not the least costly type.

Hydraulically operated valves may be moved either by water motor or by piston displacement in a hydraulic cylinder. Many valves of the Board of Water Supply were of the motor type; distance control is not feasible. The drawback to hydraulic operation is the liability to failure due to freezing.

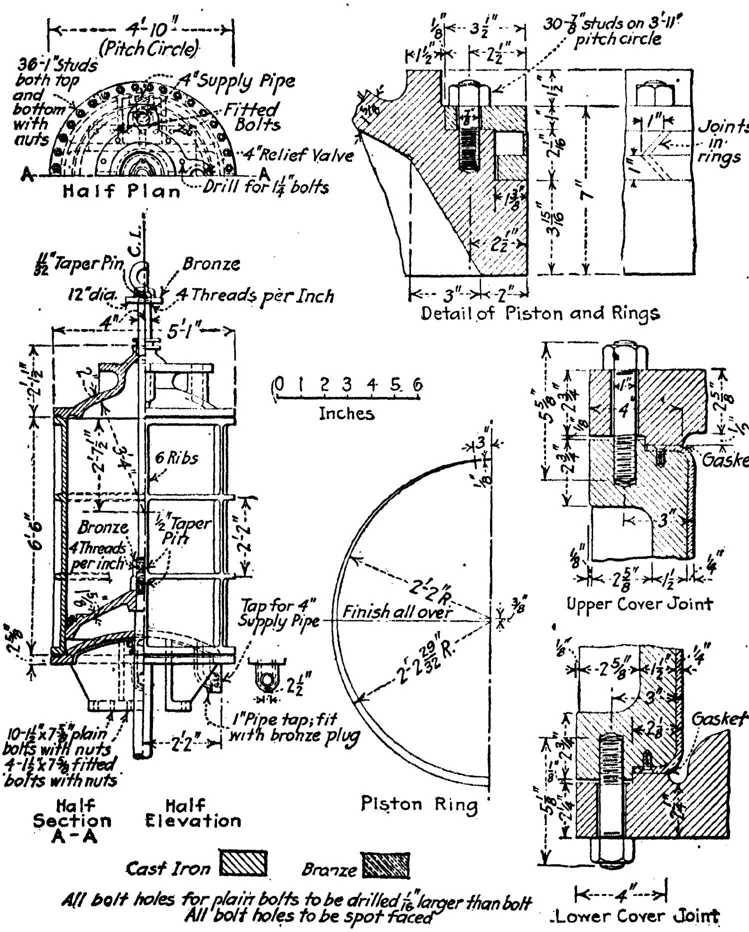


FIG. 188.—Hydraulic cylinder for large gate valves.
(Catskill Aqueduct.)

Existing valves cannot be converted to hydraulic operation. Automatic control may be fitted to cylinder valves. Hydraulic cylinders have to be ordered specially in accordance with the operating conditions to be met. Hydraulic operation was preferable until numerous difficulties with early electrical equipment were overcome.

Hydraulic-lift Valves. *Liquids in Cylinder*. Investigation has shown some form of mineral oil having a low cold test preferable to any mixture of glycerine or alcohol and water, or any brines used in refrigerating work, as

former do not remain stable in composition, and latter rapidly corrode metal. At Spot Pond gate house, Boston, ordinary ice-machine oil (zero cold test paraffin oil, Standard Oil Co.) is used. This begins to solidify at −2 or −3°F.; it is not suitable for exposed positions, but has answered perfectly in gate houses, where temperature is kept above zero. For very severe exposures, such as hydraulic cylinders on the hoisting mechanism of Charlestown bridge, Boston, a preparation of Russian petroleum, "Hydrol"* (Davies, Rose & Co., Boston), has been satisfactory since 1900, being subjected to temperatures of −10°F. or less. According to tests of Boston Bridge Dept., at −24°F., this oil has consistence of lard. In addition to antifreezing qualities, it is a good lubricant, and will not corrode metal. Hydrols is pure hydrocarbon, colorless, tastless, odorless, non-oxidizable, neutral. Sp. gr. at $69_o =$ 0.865. Does not solidify at 0°F.; at −5°F. it is viscid, slow flowing. Frankfort arsenal tests of viscosity: 0°F., 125.6 sec.; 75°F., 68.6 sec.; 100°F., 42.4 sec. It is difficult to get oiltight joints on hydraulic cylinders. Rubber packing and all ordinary pipe-joint compounds are attacked by oil. Oil or shellac serves best for getting tight screwed joints; paper or leather saturated with a mixture of glue and glycerine, for packing flanged work.

Requirements for hydraulic cylinder oil are: (1) Absolute neutrality, to prevent deterioration of metal; (2) freedom from resins or gums, which might clog passages when solidified by oxidation; (3) ability to remain liquid at 0° F.; (4) sufficient body to be retained in cylinders without excessive protection against leakage, and at same time low enough viscosity to flow freely. No vegetable oils answer above requirements; they contain too much acid, and thicken and solidify at too high temperatures. Animal oils have about same objections. Mineral oils are best; many of these are in the market. U. S. Government specifications for "Hydrolene," a standard product of many refineries, used for recoil cylinders of large guns, require: (1) neutrality at ordinary temperatures and at 150°F.; (2) freedom from ash and saponifiable oil; (3) no trace of decomposition at 200°F.; (4) sp. gr. of 0.835 to 0.87 at 60°F.; (5) cold-test point (where flow ceases) to be below 0°F. Viscosity (by Saybolt Standard Universal Viscosimeter, used by Standard Oil Co.), 65 sec. plus or minus 10 sec. at 70°F., and preferably to vary as little as practicable from this between limits of 30 and 100°F., but not to be greater than 145 sec. at 30°F., nor less than 43 sec. at 100°F. (Viscosity is generally determined by time required for a given amount to flow through a standard orifice, as compared to water, sperm oil, or rapeseed oil.) Hydrolene will expand through heating. It will not freeze at −15°. It discolors steel of guns after 12 months' contact, but the metal is not otherwise attacked.

Leakage in Hydraulic Cylinders. Tests at Coffin Valve Co. on 52-in. hydraulic cylinders for 60-in. gate valves, Catskill aqueduct (see Fig. 188) for leakage past the piston rings, consisted in placing a cylinder horizontal with the upper head removed; pressure was put behind the piston, which was prevented from moving. Under pressure of 50 lb. per sq. in., leakage was 0.72 gal. per min.; 75 lb., 0.97 gal.; 85 lb., 0.86 gal. Reduction of leakage with increase of pressure from 75 to 85 lb. appears to be due to pressure back of the rings forcing them out to make a tighter contact with the cylinder wall; 75

* Gave satisfaction, but could not be obtained during the war (Tinkham).

per cent. of the leakage came from the split in the outer ring; no special effort had been made to have a tight joint here; the space between abutting ends of the ring was 0.010 to 0.0105 in. Some refinement in the joint design might reduce the leakage 30 per cent. That the piston rings were distended under pressure was shown by introducing a 0.004-in. feeler for an arc 6 in. long opposite the split, when under no pressure; under pressure the feeler could be pushed in only ¼ in. (width of ring, 1 in.) (see detail, Fig. 188).

Friction in Hydraulic Cylinder. Tests were also made to find pressure to start the piston alone (no pressure on the plug). The cylinder was placed horizontally and the head of water measured in an attached standpipe; 3-ft. head moved the piston; 950-lb. pressure moved an unbalanced load of 3500 lb., giving a coefficient of starting friction of 0.27. The piston (spring) rings were designed to exert 4 lb. pressure per sq. in. of cylinder walls. The test proved this. On such a basis, the total pressure necessary to move the piston would be 352 lb., or a unit pressure of 0.17 lb. per sq. in. over the area of the piston.

AIR VALVES*

Use on Pipe Lines. Some form of air vent is desirable on long cast-iron and masonry conduits and is necessary to the safety and successful operation of pipe lines of wood and steel. Air becomes entrained in the water and accumulates at summits and abrupt changes of grade; and sometimes at horizontal bends in a level pipe with minor depressions to avoid sewers. Air pockets have stopped flow as effectively as valves.[15] Borden[16] observed that accumulations are greatest at summits nearest hydraulic grade line, are greater in summer than in winter, in pumping lines than in gravity lines, and at summits nearer the reservoir. To relieve these accumulations, "pressure" or "float" air valves are employed. On some early lines, manually operated valves were used. The 30-in. conduit for Syracuse, N. Y., is patrolled every 48 hr., at which times the 4-in. valves are manually opened for expulsion or air. Steel and wooden pipe lines, to facilitate passage of air to relieve unbalanced pressure tending to rupture the pipe when being filled or emptied, require "air and vacuum" (also called "automatic poppet") valves, placed near summits, and sometimes combined with the float air valves. The importance of air and vacuum valves was shown in the failure of the steel pipe at Portland, Ore. (1911) under test (see p. 312). Air valves help to render a steel pipe "foolproof." Better too many than too few, as thousands of dollars' worth of property are protected by them.[17] Cast-iron lines are equipped with float air valves, but no air and vacuum valves.

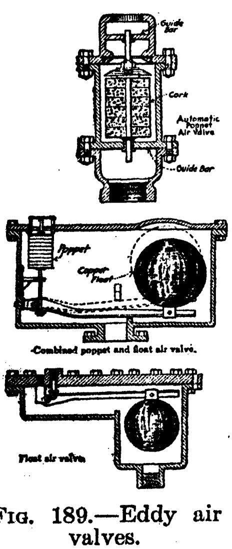


FIG. 189.—Eddy air valves.

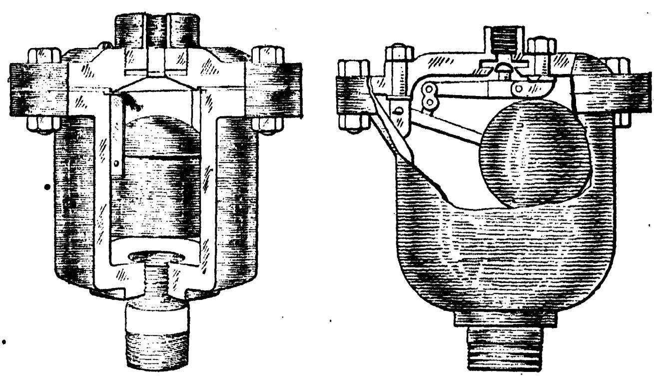
Design. Sizes and locations can be determined only by computations, a study of the profile, and investigation of existing conduits.† Hydraulic

* Makers include Water Works Equipment Co., Coffin Valve Co., Ludlow Valve Co., Eddy Valve Co., Multiplex Mfg. Co., Berwick, Pa.; Simplex Valve and Meter Co.

† See also "Hydraulics of Pipe Lines," by W. F. Durant (Van Nostrand, 1921); "Factors Governing Air Inlet Valves on Pipe Lines," D. C. Henny, *E. N. R.*, Aug. 19, 1926, p. 294; "Pipe Line Inlet Air Valves," by Ledoux, Water Works, July, 1926.

Power Comm.[11] recommends that air flow be calculated on basis of half the difference of pressure that the pipe can safely withstand without collapse. According to tests by Carman and Carr,[18] this difference = 50,200,000 $\left(\frac{t}{d}\right)^3$, where t = plate thickness, in.* and d = pipe diam., in. Hazen's rule ("American Civil Engineers' Handbook")† is 1 sq. in. of area for each ft. diam. of pipe; also $\frac{1}{64}$ area of pipe. In East Jersey Water System, early steel pipes were so proportioned that a break at the most dangerous point could at no time or place cause more than $\frac{1}{2}$ atmosphere or vacuum inside the pipe. This resulted in an average of 38 sq. in. of air valve area on each 1000 ft. of line.[19]

Air-valve vaults are generally provided to give access for inspection. In city streets, where the pipe is in shallow trench, conditions often require tapping on the horizontal diam. for air-and-vacuum valve. Between air



Automatic air and vacuum. Pressure.

FIG. 190.—Crispin air valves.

valve and pipe, place a gate valve. A vent should connect air valve to the outer air. As an air valve on horizontal center line cannot remedy air accumulations at the top of the pipe, these must be controlled by a pressure valve tapped into the top of the pipe. In shallow vaults, there is liability to freeze; in northern New Jersey the vaults are filled with straw during the winter and cleared out each spring.

Air Valves Blown Shut. In purchasing air and vacuum valves investigate their capacity to pass air through at great velocity. Some valves can be blown shut with very little air pressure because the floats are so arranged that the escaping air from the main strikes them underneath, defeating the purpose. Floats and balls should be hard rubber, like meter disks. Figure 189 shows three types of air valves.

Negative-pressure air valves‡ can be operated by increasing the water pressure (by partial closing of any gate downstream) and thus air can be released before it has accumulated in the quantities required to operate an automatic air valve, and which, during its accumulation, constitutes an

* Cold-drawn seamless steel tubes.
† Fourth edition (Wiley, 1920), p. 1246.
‡ Water Works Equipment Co.

impediment to flow. Negative-pressure valves are an advantage over the manually operated in that a valve throttled several miles away will cause them to open; time and money are saved.

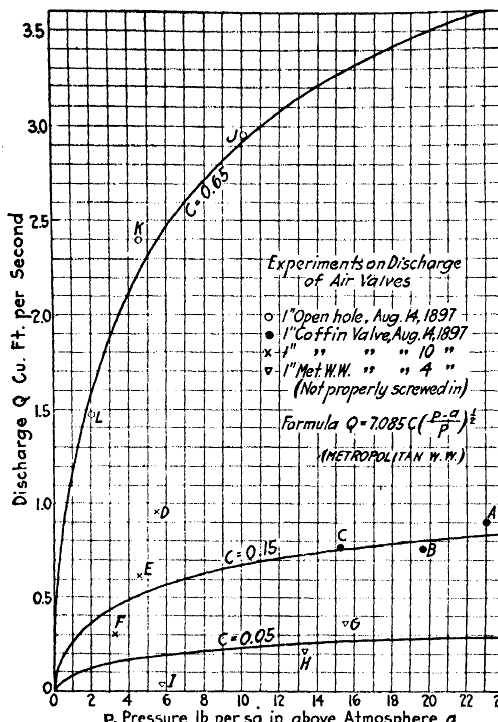


FIG. 191.—Discharge of air valves.

PRESSURE-REGULATING VALVES

Pressure Regulation. The problem of satisfactory pressures is often complicated by topographical conditions; efficient fire pressure on high lands giving excessive domestic pressure in low lands. One solution is separate mains and a pumping station for each district; another is the use of pressure regulators. If a town is supplied from a source so high that direct pressure would be injurious to mains and services, pressure-regulating apparatus may be inserted in the supply main, or preferably on a by-pass, valved at either end, to provide proper domestic pressure. Such appliances are also used on old weakened mains to reduce high pressure to a safe pressure.

Pressure regulators,* to give satisfactory service, should be of simple design containing few parts and no delicate mechanism; should be strong, durable, and reliable; all parts liable to rust should be made of non-corrodible material. Regulators should not close suddenly enough to cause water-ram in the pipe lines. Troubles are sometimes experienced from water-hammer, as

H. Mueller Mfg. Co., New York City; Watson Governor Co., Boston, Mass. (Donne v plex Valve & Meter Co., Philadelphia, Pa.; and Mason Regulator Co., Boston, Mass.

at Barre, Vt., where a head was reduced from 272 to 3 ft.[32] The apparatus should be operated entirely from the low-pressure side to avoid the effects of pressure fluctuations upstream. Valves too sensitively balanced operate under every pressure fluctuation and get considerable wear; ordinarily, operation under a pressure change of 2 to 4 lb. gives satisfactory service. In gritty water, a sand catcher just above a regulator chamber diminishes wear. Pressure-regulating valves cannot be used where the flow alternates in direction, as in a reservoir supply pipe which also serves as discharge.[20] Pressure regulators open wide in case of a break in the main being supplied and let water through, since their purpose is to maintain pressure on the downstream side.

Like other automatic devices, pressure regulators need some attention, if they are to be kept in good condition; therefore, they should be inspected at

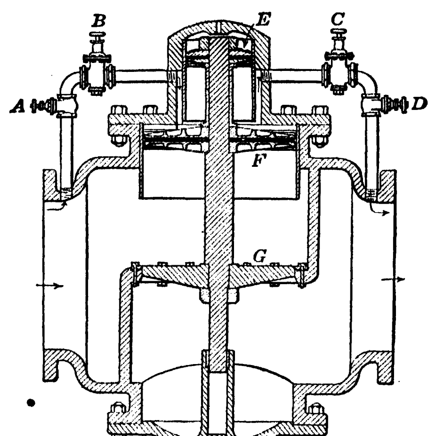


FIG. 192.—Ross pressure regulator.[20]

suitable intervals by a competent mechanic. In a large system some regulators will have much harder service than others and should have more frequent attention. Foster type of regulators were used on some New York City work with good results. Chambers should afford room for removing stem.

Choice and Location. If pressure regulators of very much too large capacity are installed, they are likely to be unsatisfactory, because of working so near the closed position and with such small movements. Multiple connections of small regulators (a battery) rather than one large regulator are sometimes advocated. A pressure regulator should always be located on a by-pass so that it can be cut out for repairs. Never place regulators at summits, as the entrained air may interfere with their operation.

Ross Pressure-regulating Valve (Fig. 192). A stem carries the pistons *E* and *F*, and disk valve *G*, which seats by an upward movement, on a leather collar. Areas of valve *G* and piston *F* are equal, preventing any tendency of the piston to move up and down. The actual reducing of pressure is done by valves *B* and *C*, which act on the check-valve principle; for instance, if the

desired outlet pressure is exceeded, the upward pressure under valve *G* causes an increase of pressure in the controlling chamber; this closes or diminishes the flow through the regulator *B* and opens the relief valve *C*, allowing the pistons to rise and close the valve. When the outlet pressure falls, the pressure in the chamber is reduced, the regulating valve opens, and the relief valve closes, allowing the pistons to fall, thus opening the valve and increasing the pressure on the delivery side. The pressure at which the valve will open and close is determined by the hand wheels on *B* and *C*. This valve has few moving parts; no springs on the main valve; the seat, of the piston type, obviates chattering under irregular delivery when the valve is nearly closed; the valve is easily adjusted within its working limits, and has given satisfaction on the high-pressure fire service in New York and Brooklyn, holding the pressure with great uniformity and requiring the minimum of attention. The action of the valve is affected to a certain extent by the upstream pressure; should this fall much below the pressure for which the auxiliary valves are set, the main valve would either close or merely maintain reduced pressure on the downstream side; action is dependent on diaphragms and springs, introducing an element of maintenance and uncertainty, although experience shows but little trouble. The main valve seat and dash-pot plunger are leather or rubber packed; repacking necessitates cutting the valve out of service. In an installation of 20 regulators (6 to 16 in.) in an eastern city, leathers last about 2 years; with mild service, leathers last indefinitely.

A Ross regulator at Passaic, N. J.,[41] has operated continuously and satisfactorily since installation in 1908, according to Superintendent Cuddeback. This 16-in. regulator has passed a widely varying quantity (0 to 8 mgd.) with upstream pressure of 100 lb. per sq. in., downstream, 30 to 40 lb., delivering from a 51- to a 30-in. main. Ross valves can operate when axis is 45° from vertical, but repacking is extremely difficult.

Table 126. Dimensions and Weights of Ross Pressure Regulators

Size, in.	Face to face dimensions, in.	Approximate weight, lbs.
2	$6\frac{7}{8}$	50
3	$8\frac{7}{8}$	80
4	14	250
6	$17\frac{3}{4}$	450
8	$23\frac{1}{2}$	600
10	$24\frac{1}{2}$	850
12	30	1250
16	37	1850
20	$41\frac{3}{4}$	3400
24	48	4000

Miscellaneous Types. Pressure-controlling valves close when the pressure attains an assigned limit, and stop the overflow of reservoirs. Pressure relief valves open automatically to ample area when the pressure reaches an assigned limit, and relieve the stress on the pipe line. Float valves are actuated by a ball floating on the surface of the reservoir, which actuates the

valve when the water reaches an assigned level. The makers of pressure-regulating valves, mentioned on p. 448, in general, make these valves also. A Dean controlled valve can also be operated by a reservoir float.

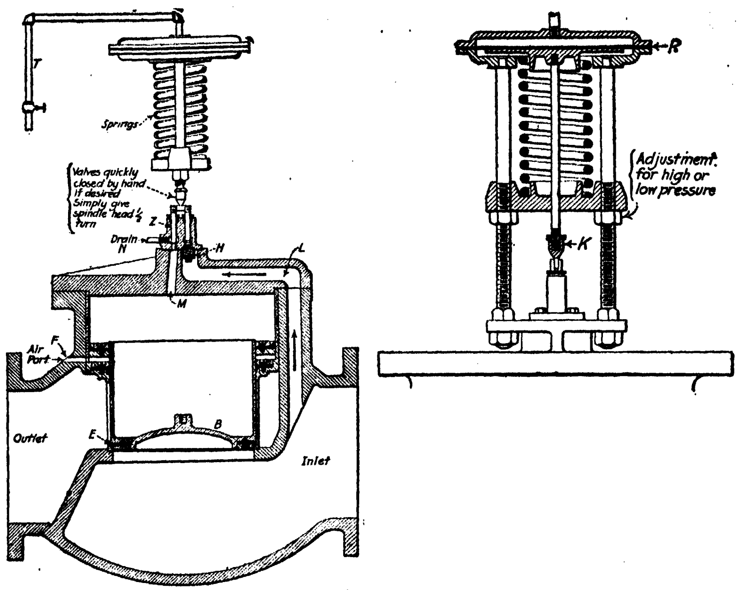


FIG. 193.—Anderson automatic controlling altitude valves.*

OTHER VALVE FORMS

Check Valves† for Protecting Centrifugal Pumps. When a foot valve is used on suction pipe of a centrifugal pump pumping to great height, or discharging through a long pipe, severe strain is put on the pump every time it is

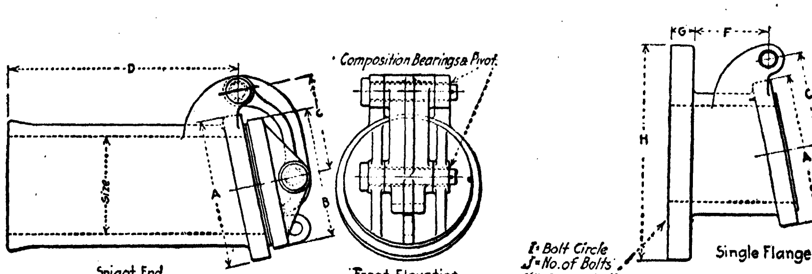


FIG. 194.—Coffin double-pivot flap valve. (See Table 128.)

stopped, owing to "water-hammer," caused by suddenly closing the foot valve; this is often large enough to crack shell of pump. This trouble can best be prevented by placing a check valve‡ immediately at the discharge

* Recent designs embody an improvement which enables all adjustments to be made on one nut.

† For makers, see first foot-note, p. 436.

‡ Omission of check valve may result in a wreck, when the power fails. See *E. C.*, Sept., 1925, p. 566.

flange of the pump. Large waterways must be provided through check valve. The Smolensky noiseless check valve,* the Golden-Anderson, double-cushioned, check valve, and others are designed to eliminate water-hammer.

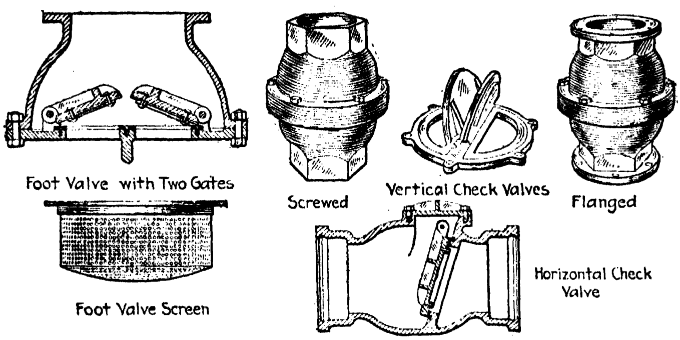


FIG. 195.—Foot valves and screens, and check valves.[38]

Table 127. Dimensions of Check Valves, Bronze Mounted[38]
See Fig. 195

Size, in.	Hub		Flange	
	End to end, in.	Laying length, in.	Diam., in.	Face to face, in.
3	14¾	8¾	7½	10
4	18⅜	11⅜	9	12
5	19¾	11¾	10	13
6	23	15	11	15
8	27	19	13½	18
10	28½	20½	16	20
12	34	26	19	26
14	34	26	21	27
16	38	30	23½	30
18	39	31	25	32
20	43	35	27½	36
22	48	40	29½	40
24	54	46	32	44
30	68	59	38¾	56

Check valves for mill supplies were insisted on in old practice, to safeguard cross-connection to municipal supply from the lower-pressure auxiliary service to the mill, which was of inferior quality. Lack of tightness in these valves has in many instances led to contamination of the domestic supply as at Franklin Furnace, N. J., in 1922. See p. 403. System of double-check valves with a gate valve upstream and downstream has been devised by Assoc. Factory Mutual Fire Ins. Co. (see *J. A. W. W. A.*, Vol. 8, 1921, p. 222). If such connections must be tolerated, Saville prefers this system.[22] In 1920, Hartford decreed that all connections between city supply and other sources be abolished, including those protected by double-check valves.[23] The regulations of many State Departments of Health prohibit cross-connections (see Washington (State) rules, *E. N. R.*, July 10, 1924, p. 62).

* Smolensky Valve Co., Cleveland, Ohio.

Table 128. Dimensions of Coffin Standard Single-pivot and Double-pivot Flap Valves

See Fig. 194

	Spigot end							Flange dimensions		
Size	A	*B*	*C*	*D*	*F*	*G*	*H*	*I*	*J*	*K*
4″	6 1/16″	5 7/16″	4 1/16″	12″	3 1/16″	15/16″	9″	7 1/2″	4	5/8″
6″	8 1/4″	7 3/8″	5 5/16″	12″	4″	1″	11″	9 1/2″	8	5/8″
8″	10 1/2″	9 13/16″	6 1/2″	12″	4″	1 1/8″	13 1/2″	11 3/4″	8	5/8″
10″	12 3/4″	12″	7 3/4″	12″	4 1/4″	1 3/16″	16″	14 1/4″	12	3/4″
12″	15″	14 1/4″	9″	12″	4 7/8″	1 1/4″	19″	17″	12	3/4″
14″	17 15/16″	16 9/16″	10 1/4″	12″	5 1/4″	1 3/8″	21″	18 3/4″	12	1/8″
16″	19 5/8″	18 5/8″	11 5/16″	12″	5 1/4″	1 7/16″	23 1/2″	21 1/4″	16	7/8″
18″	21 7/8″	20 7/8″	12 1/2″	12″	5 1/2″	1 9/16″	25″	22 3/4″	16	1″
20″	23 15/16″	22 15/16″	13 3/4″	12″	6″	1*M*″	27 1/2″	25″	20	1″
24″	28″	27″	15 3/4″	12″	6 1/8″	1 7/8″	32″	29 1/2″	20	1″
30″	34 5/8″	33 3/8″	19 3/8″	14″	6 3/8″	2 1/8″	38 3/4″	36″	28	1 1/8″
36″	41 1/4″	39 3/4″	23″	14″	7 1/8″	2 3/8″	46″	42 3/4″	32	1 1/8″

Note.—Dimensions *A*, *B* and *C* are same for all types. Dimensions *F*, *G*, and *H* are same for single- and double-flange types.

Cylindrical Gate Valves. Dam on Magalloway River, Maine, has 8-ft. undersluices terminating in vertical risers, openings of which are controlled by cylindrical gate valves, such as are used on certain types of turbines. With full opening, under 55-ft. head, the valves each discharge 2000 c.f.p.s. The following advantages are claimed: quick manipulation; close regulation of discharge; operation by one man; maximum discharge area with minimum head permitting drawing reservoir to lowest practical level, at same time maintaining large discharge through gates; rugged construction. Partial tests show coefficient of discharge of 0.89.[39]

Controllers for filter plants, manufactured by Builders Iron Foundry, Simplex Valve and Meter Co., and others, are a combination of Venturi meter and butterfly valve, the waterway of which is controlled by the flow through the Venturi (see p. 459); so that the rate of filtration is controlled.

Needle valves are designed to offer less resistance to flow and to operate with less power, under the excessive head usual on reservoir outlets. Ensign* and Johnson† valves are the best known.

The Johnson hydraulic valve, introduced in 1910, is designed, mechanically as well as hydraulically, so that it can be built in sizes to which no other type of valve could be applied. It (Fig. 196) consists, in general, of a circular body surrounding an internal cylinder, closed at one end and connected to the body by radial ribs, in which a pointed plunger or needle operates, making contact with a seat in the neck of the body to close the valve. The pressure used to move the plunger is the pipe-line pressure in the body of the valve itself. When space *A* is open to line pressure, and *B* is open to atmosphere, the valve closes; reversing these pressures opens the valve. The flow through the valve at all positions of the plunger is smooth and free from disturbance. Friction losses are small. Johnson valves are in use and under construction in sizes up to 20 ft. in diam. Some larger valves operate under heads up to 1000 ft.; some smaller valves have been tested under heads up to 3000 ft. There are

* For design at Elephant Butte, see *E. N.*, Feb. 22, 1917, p. 306.

† Johnson valves used at O'Shaughnessy (Hetch-Hetchy) dam. Furnished by Wm. Cramp & Sons, Ship & Engine Bldg. Co., Philadelphia.

no sliding contacts under pressure, as in other valves. The plunger in operation is unbalanced only enough to move it. The internal cylinder in which the plunger operates is made of cast iron for all pressures, as it is subject to compressive stress only. Only small sizes and valves which are seldom operated are protected with a full bronze lining. No matter how large the valve or how high the pressure, the valve is always tight when closed. The plunger has a renewable seat ring. The higher the pressure in the valve the higher the pressure of the plunger against the seat, but there is no rubbing of one seat on the other and hence there is no wear. Distortion, which is a common cause of leakage in other valves, is entirely absent on account of circular shape.

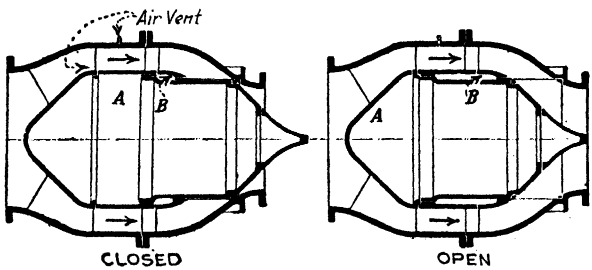


FIG. 196.—Johnson hydraulic valve (patented).

Reservoir outlets may be controlled by gate valves, sluice gates, needle valves, or other special forms. (See "High Pressure Gates in Dams Reviewed," by D. W. Cole, *E. N. R.*, Nov. 14, 1918, p. 880; "Some Experiences with Large-capacity Reservoir Outlets," by J. M. Gaylord, *E. N. R.*, Nov. 21, 1918, p. 945; and also Chap. 6, p. 105. For sector gates and other spillway controls, see Chap. 6, p. 104.

Quick-operating valves have a lever attached to the valve stem. They are used for blow-offs of reservoirs and elsewhere if sudden opening or closing is imperative.

SLUICE GATES*

Stock Sizes. Gates have not been standardized nor generally carried in stock. Makers, such as Coffin Valve Co., Boston; Chapman Valve Mfg. Co., Springfield, Mass.; Coldwell-Wilcox Co., Newburgh, N. Y.; Michigan Valve Co., Detroit Mich.; and Hinman Hydraulic Mfg. Co., Denver, Colo., have patterns for a wide variety of sizes of square, rectangular, and circular gates, ranging from 10- by 10-in. to 216- by 87-in., and up to 108-in. diam. designed to meet various conditions as to pressure, etc. For small works selection can usually be made from these stock patterns; for large works it is sometimes necessary or economical to modify existing designs or prepare new ones. Rising stems should be used wherever practicable. Ball-bearing, roller-bearing, and other friction-reducing types of operating stands or hoists are obtainable, also electric and hydraulic motor drives and hydraulic lifting cylinders. On outlet of Black Canyon dam, sluice gates are placed in the pipe line, similar to a gate valve.[32]

Ordering. The same kind of information as for gate valves (p. 436) is needed. It is important to state exactly the working head, and whether

* For Stoney and other types for spillways, see p. 104.

pressure tends to push gate away from the seat; for the latter condition, back-pressure gates are used.

Shear gates are a primitive type of sluice gate, having no guides. In practice, they should be avoided, if a backpressure is at all likely, as instances are known of backpressure causing objectionable leakage, and even twisting the shear gate so that it was useless.

Narrow Sluice Gates for High Pressure. Large sluice gates under high heads should be made narrow in proportion to the height so as to secure a reasonable pressure per lin. in. on the seats. To save head room when open, a double-hung sluice gate was used at Sudbury dam.[25]

Power to Operate. Friction tests of bronze on bronze in some large gates for U. S. Reclamation Service[26] gave

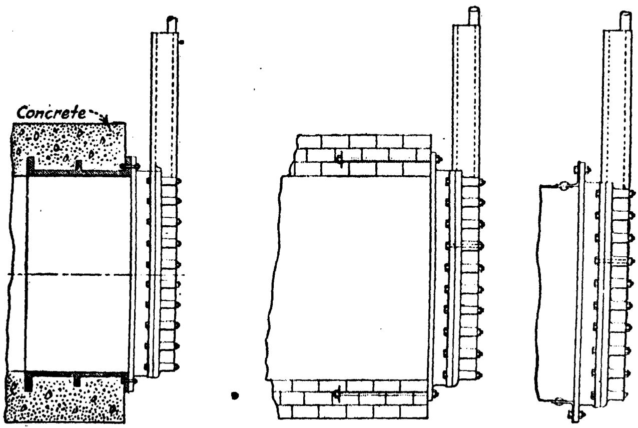


FIG. 197.—Methods of attaching sluice gates by flanges to wall castings, masonry and pipes.

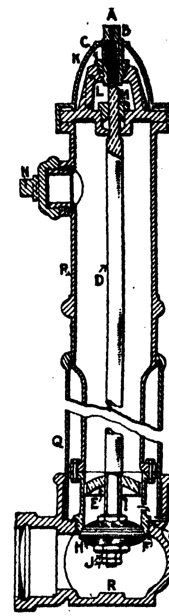


FIG. 198.—"Standard" sliding frost-case compression fire hydrant (A. P. Smith Mfg. Co.)

values of 0.38 to 0.44. Newell and Murphy[42] use 0.25 in a sample calculation.

Protection against frost, by housing and electric heating, was secured at Bassano dam[33] by a complete system of electric heating for the gates. Special precautions in the way of housing have been taken to assure that all the canal head sluices and two of the crest sluices shall be free from ice at all times.

HYDRANTS

Standard specifications were adopted by the A. W. W. A., in 1913, and N. E. W. W. A., in 1914. These are so broad that patented hydrants of manufacturers generally meet them. Only for special work are detailed specifications required.

Designs. For list of makers of valves and hydrants, see footnote, p. 436. Most large cities have their own designs; for Cincinnati standards, see *E. N.*, July 22, 1915, p. 153, and *E. N.*, Mar. 15, 1917, p. 425; Chicago standards, *E. N. R.*, May 9, 1918, p. 921. Hydrant tests in Chicago with standard makes indicated several defects in design, which were remedied in the city's new standards.[28] For high-pressure hydrants in which seawater may be used see *J. N. E. W. W. A.*, Vol. 36, 1922, p. 487 for Boston design, and *E. R.*, May 20, 1905, p. 583 for New York design.

Hydrants may be designated by the number of 2½-in. hose outlets, for which they are designed. (If a steamer connection is also provided, the hydrant would be designated, for example, "two-way and steamer connection.")

Table 129. Post Hydrant Bells. N. E. W. W. A.

Nominal pipe diam., in.	Classes	Actual outside diam. of pipe spigot, in.	Pipe sockets		"a"	"b"
			Diam., in.	Depth, in.		
6	All	7.10	7.90	3.00	1.50	1.40
8	All	9.30	10.10	3.50	1.50	1.50

Table 130. Post Hydrant Flanges. N. E. W. W. A.

Size of pipe, in.	Diam. of flange, in.	Flange thickness at edge, in.	Diam. of bolt circle, in.	Number of bolts	Size of bolts, in.
6	11	1	9½	8	¾ × 3
8	13½	1⅛	11¾	8	¾ × 3½

Where working pressure is from 150 to 250 lb., the standard for heavy flanges must be used; see p. 390.

When ordering hydrants state kind wanted; size of valve opening; depth from surface of ground to bottom of connecting pipe; size of connecting pipe; whether screw, bell, or flange end; number and size of nozzles. Send a nozzle cap as gage for thread and nut, or give exact outside diam. and kind of nozzle threads, also size and form of nut; state whether to open to right or left. Consult catalogs of manufacturers.

Requirements, Board of Fire Underwriters. For spacing, see pp. 401 and 457. Hydrants shall be inspected in spring and fall, after fire use in freezing weather, and daily during severe cold. Hydrants shall deliver 600 g.p.m. with a total loss of not more than 5 lb. between street main and outlet.* There shall be not less than two 2½-in. outlets, and also a large suction connection where engine service is necessary. Street connections shall be not less than 6 in. and gated.

Maintenance. Hydrant use for other than fire purposes should be subject to strict regulations. Typical instructions are those by George G. Earle for New Orleans.[29]

To Parties Using Fire Hydrants:

Your co-operation in seeing that the following rules for the use of fire hydrants are strictly complied with is earnestly requested:

* Loss in a 5-in. hydrant was measured by Saville[27] as scarcely anything when flowing 300 g.p.m., and varying along a nearly parabolic line to 22 lb. at 3000 g.p.m. Hydrant pitometers are made by Waterworks Equipment Co., and others.

1. Open a fire hydrant slowly, and steadily, and always enough to prevent any chattering of the main valve, even if more water is drawn than is required.

2. If the fire hydrant is to be left open, with pressure available, for self-closing or other faucet, or valve, on its outlet, for intermittent use, open it slowly, until the hydrant is full open, then release the operating nut by one turn backwards. Where a hose bib or other connection is placed on hydrant, care must be used to see that rubber gaskets are placed under the nozzle caps.

3. Close a fire hydrant very slowly, especially as the valve approaches its seat, until it comes firmly to its seat, and then release tension on valve and stem by one turn backwards of operating nut.

4. After closing a fire hydrant, see that the caps are properly replaced over its outlets.

5. Do not allow a fire hydrant to run for gutter flushing after the flow in the gutter becomes reasonably clear.

6. Use no tools to operate a fire hydrant other than a close-fitting five-sided wrench furnished by the Sewerage and Water Board. If wrench becomes worn, so that it injures the operating nut, turn it in and obtain a new one in its place.

Waterproof graphite grease applied to the valves and caps of fire hydrants forms a coating that cannot be washed off even by the rushing stream of water; it preserves the valves and cap from rust, and from sticking by "freezing." It will not gum or become rancid.

Painting fire hydrants a distinguishing color, such as orange yellow, which is more conspicuous than the conventional red or green, aids the fire department.[34]

Frozen hydrants may be successfully cleared by steam, hot brine, calcium carbide, or alcohol, or by building a fire about them.* Abuse of hydrants when used for flushing, etc. may be avoided by provision of "Sprinkling and Flushing Hydrant,"† which also eliminates the unsightly goosenecks from parks, etc.

Hydrant practice, Terre Haute Water Co.:[30] (1) 6-in. branches, with 6-in. valves, to cut out hydrant for repairs; (2) brick drain pits at hydrant base, 1 cu. ft. capacity, and open at bottom for quick draining; (3) hydrants flushed in April and September and inspected in late fall; (4) hydrants open by turning to left; (5) hydrant model available for instruction of firemen; (6) standard hydrants carried in stock; (7) defective hydrants reported to Fire Department; (8) after fire use, hydrant inspection.

Hydrant connections to mains should be at least 5 in. and should be provided with a gate valve. Some manufacturers furnish this valve with the hydrant.

Hydrant Spacing.‡ The Committee of the A. W. W. A. recommended smaller standard sizes of hydrants at frequent intervals, rather than large hydrants with numerous outlets at longer intervals. Hydrant spacing should be governed largely by character and value of the property protected. In thickly built mercantile and manufacturing sections, hydrants should be about 200 ft. apart, and not more than 500 or 600 ft. apart at the maximum anywhere. In other words, it should be possible to concentrate a sufficient

* *Report* of Comm., *J. A. W. W. A.*, Vol. 7, 1920, p. 751.
† One type is made by H. Mueller Mfg. Co.
‡ See also p. 401 and 456.

number of effective streams on any building which may burn, and for this end lengths of hose should not be greater than 250 to 350 ft.

Since the loss of head per 100 ft., when delivering 250 g.p.m. through 2½-in. hose, is about 18 lb., and a nozzle pressure of 45 lb. is required, a hydrant-flow pressure of 100 lb. will supply but 300 ft. of hose. Conard[40] justifies spacing as close as 100 ft. in certain districts on basis of saving in cost of mains and purchase of hose lengths. Post hydrants should be about 14 in. from curb so that passing vehicles will miss the nozzle. Vehicles damage 400 hydrants per year in New York city.[43]

Subsurface hydrants* are in favor in Australia and other warm regions where snow never occurs to hide them.[31] Insurance interests do not endorse them; they were in use in Indianapolis, New Bedford, and elsewhere and were found unsatisfactory, due to water-hammer, snow, and mud.[35] In Boston they were an advantage on the narrow sidewalks in the narrow, crowded, old streets, as they offered no obstruction to traffic.

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* Also termed "flush hydrants."

CHAPTER 21

SERVICE METERS

Advantages. The placing of recording meters at all strategic points on supply lines (pumping stations, reservoir outlets, connections to other municipalities, etc.) is considered good practice. The merits are twofold: (*a*) The accounting department has a record of relative uses and can allocate costs accordingly; (*b*) consumers and the operating department can use the records to locate excessive use and take steps to control it. The advantages of metering each customer's service are often obscured when a partisan press attempts to give a political status to the meter question. Metering should be considered an investment in the broad sense that it defers the expenditure of larger sums for waterworks enlargements. For statistics showing benefits of metering, see p. 468.

Types.* For large conduits, flow may be gaged by various means. (Venturi meters are discussed below.) Where conditions are such that altering of line to incorporate the tube is undesirable, a flow nozzle,† or an orifice plate‡ (with recording device) may be inserted, or some form of recording pitot tube§ utilized. Colorado Fuel & Iron Co. uses current meters, introduced through valved risers. New England Power Co. utilized a reducing bend as a Venturi meter, to avoid shutdown incidental to incorporating a tube.[1]

Lea V-notch recording meter measures and records graphically flow of water over V-notches. It is actuated by a float in a chamber directly beneath the instrument; is applicable to measurement of boiler-feed water, discharge from condensers, pump discharges, streams, irrigation ditches, canals, acids, and other fluids, but has an inherent and unavoidable loss of head. Made by Yarnall-Waring Co., Philadelphia. Advantages are: continuous measurement; no mechanism in contact with water; same instrument used for measuring large or small quantities by using notch plates of various angles, or two or more 90° notches. Guaranteed by makers to be within $1\frac{1}{2}$ per cent. of absolute accuracy by weight; also that average error due to variations in temperature over range of 50°F. will not exceed 0.5 per cent.

VENTURI METERS

Description. The Venturi meter consists of tube and registering instrument. The tube, incorporated in the pipe line, has two conical frustums (the upstream being ordinarily shorter) connected by a cylindrical throat piece. Pressure piping from throat and upstream end of tube transmit pressures to

* See also p. 463.
† General Electric Co.
‡ Republic Flow Meters Co., Chicago.
§ Manographs are put out by Water Works Equipment Co., New York City; photopitometers by Pitometer Co., New York City.

a mercury column in the register. Readings depend solely on the relative pressures; there is no mechanism in path of flowing water, which is an advantage. Tubes are made in stock sizes (see Table 131, p. 462), usually of cast iron. For special cases they may be reinforced concrete, wood, or other material. It is essential that interior of throat piece be non-corrodible. There are several kinds of instruments, some indicating differences of pressures at entrance and throat, others registering or recording the flow in various units; some combine indicator, register, and chart recorder. For determining the total quantity for the period covered by a dial chart, or any fraction of such period, the Builders Iron Foundry furnishes a special planimeter.

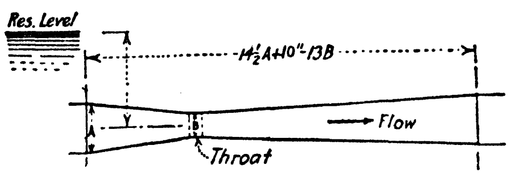

FIG. 199.—Relation of length and diameter of Venturi meter.

Hydraulic principle is based upon the relation between velocity and pressure of fluids flowing through a continuous pipe of varying cross-sectional area. This principle was first stated in 1797 by J. B. Venturi, an Italian philosopher, who had observed that fluids discharging through an expanding nozzle exert a sucking action at the small diam. which diminishes as diam. increases toward outlet. In 1887, Clemens Herschel, after elaborate experiments, adapted the principle to accurate measurement of fluids by means of the Venturi meter, now used for water, oil, steam, gas, sewage, and other fluids. In passing from full section to throat, velocity is gained at expense of pressure. Pressure at throat is thus less than at upstream entrance; water passing through a tapering tube has been found by experiment to be nearly equal to the theoretical discharge from the throat opening at a velocity corresponding to difference of pressure between throat and entrance.

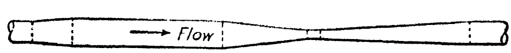

FIG. 200.

Tube Proportions. The most advantageous divergence for the downstream tube, and probably for any diverging tube, to obtain the best effect in developing Venturi action, is 1-in. increase in diam. for each 11¼-in. increase in length, equivalent to an angle at vertex of cone of a trifle more than 5° 5′. Reliable experiments indicate that throat diam. should lie between one-third to one-half diam. of upstream end; for special requirements, this rate is sometimes increased to three-fourths. The range of throat velocities usually adopted is between 25 and 38 ft. per sec. Wherever a sufficient ratio between diam. of upstream end and throat cannot be obtained in the ordinary sizes of pipe, the pipe may be increased in diam. for the approach to the meter (Fig. 200).

Directions for Installing Venturi Meter Tubes.* The piping should have a pronounced up or downgrade, and should contain no summits nor depressions where air or silt might collect; and a valve (or corporation cock) should be placed on each pressure pipe close to tube. If a summit or depression is absolutely unavoidable, a blow-off valve should be provided at such point. All joints must be perfectly tight. Piping and registers must be properly protected from frost. J. W. Ledoux[2] advises use of a compensating device, such as an air chamber, to prevent overregistration on lines subject to pump pulsations.

At Chestnut Hill pumping stations, Metropolitan Waterworks, air chamber, shown in Fig. 201, inserted between meter and boiler-feed pump (single-acting, with $2\frac{1}{2}$-in. plunger, 24 r.p.m.) "proved entirely successful in reducing pulsations due to pump so that accuracy of meter is not affected." Water in

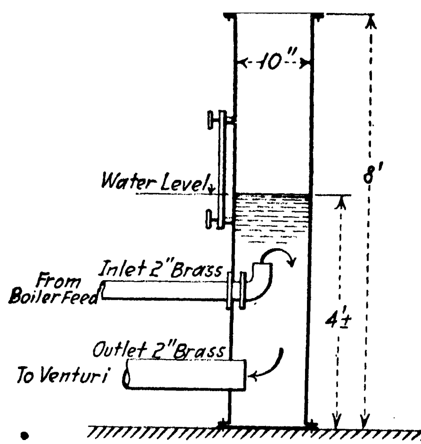

FIG. 201.

chamber is kept at level which experimentation shows gives best results. The Venturi meter is 2 in. in diam. with $\frac{3}{4}$-in. throat, with an average registration of 29,000 gal. per 24 hr.[3]

Losses through a Venturi Meter. Venturi meter tubes do not introduce appreciable friction. Usually the normal rate of discharge through the pipe line is about one-half the measuring capacity of the meter. At this rate increased friction due to the meter tube is only $\frac{1}{4}$ lb. per sq. in. Even when the meter is operated at its highest capacity, the friction amounts to only 1 lb. per sq. in. A general rule for head lost seems to be 0.11 velocity head at throat.

$$\text{Discharge in cfs.} = C\frac{\pi d_2^2}{4}\sqrt{\frac{2g(h - h_2)}{1 - \left(\frac{d_2}{d}\right)^4}}$$

in which d and h = diam. and piezometric head in feet, respectively, at A (Fig. 199); d_2 and h_2 the same at B. C is a coefficient, generally taken at 0.95 to 0.99. W. S. Pardoe[4] presents analysis substantiated by tests, whereby the coefficient may be calculated closely.

* Builders Iron Foundry, Providence, R. I.

Table 131. Standard Venturi Meter Tubes[27]
Capacities, Lengths, and Weights

Inlet and outlet diam., in.	Length* Ft.	Length* In.	Measuring capacity, thousand gal. per 24 hrs. Min.	Measuring capacity, thousand gal. per 24 hrs. Max.	Approx. weight, lbs.
2	1	11⅞	4	51	50
	1	10¼	6	73	
	1	7	10	130	
2½	2	4⅝	7	100	85
	2	3	10	130	
	1	11¾	16	203	
3	2	11	10	130	110
	2	7¾	16	203	
	2	4½	23	293	
4	4	3¾	16	293	160
	3	10⅞	26	343	
	3	6	40	520	
5	5	1⅜	26	343	275
	4	8½	40	520	
	4	2	63	813	
6	5	11	40	520	450
	5	4½	63	813	
	4	10	90	1,170	
8	7	6¼	76	983	700
	6	11¾	106	1,373	
	6	2	160	2,080	
10	9	4¾	106	1,373	1,100
	8	7	160	2,080	
	7	6	250	3,250	
12	11	0	160	2,080	1,550
	9	11	250	3,250	
	8	10	360	4,680	
14	12	10½	203	2,633	2,200
	11	6¼	331	4,298	
	10	2	490	6,370	
16	14	5¾	276	3,583	3,000
	13	1½	423	5,493	
	11	6	640	8,320	
18	16	1	360	4,680	3,700
	14	5½	563	7,313	
	12	10	810	10,530	
20	17	11½	423	5,493	4,750
	16	4	640	8,320	
	14	2	1,000	13,000	
22	19	10	490	6,370	5,700
	17	8	810	10,530	
	15	6	1,210	15,730	
24	21	2	640	8,320	6,800
	19	0	1,000	13,000	
	16	10	1,440	18,720	
26	23	0½	723	9,393	8,300
	20	4	1,210	15,730	
	18	2	1,690	21,970	
28	24	11	810	10,530	9,600
	22	2½	1,323	17,193	
	19	6	1,960	25,480	
30	26	3	1,000	13,000	11,000
	23	0	1,690	21,970	
	20	10	2,250	29,250	
32	28	1½	1,103	14,333	12,700
	25	5	1,690	21,970	
	22	2	2,560	33,280	
34	30	0	1,210	15,730	14,300
	26	9	1,960	25,480	
	23	6	2,890	37,570	
36	31	4	1,440	18,720	16,500
	28	1	2,250	29,250	
	24	10	3,240	42,120	
38	33	9	1,440	18,720	18,700
	29	5	2,560	33,280	
	26	2	3,610	46,930	
40	35	1	1,690	21,970	20,900
	30	9	2,890	37,570	
	27	6	4,000	52,000	
42	36	5	1,960	25,480	23,700
	32	1	3,240	42,120	
	28	10	4,410	57,330	
44	37	9	2,250	29,250	26,400
	34	6	3,240	42,120	
	30	2	4,840	62,920	
46	40	2	2,250	29,250	29,700
	35	10	3,610	46,930	
	31	6	5,290	68,770	
48	41	6	2,560	33,280	33,000
	37	2	4,000	52,000	
	32	10	5,760	74,880	
50	42	10	2,890	37,570	36,900
	38	6	4,410	57,330	
	34	2	6,250	81,250	
52	45	3	2,890	37,570	40,700
	39	10	4,840	62,920	
	35	6	6,760	87,880	
54	46	7	3,240	42,120	44,600
	41	2	5,290	68,770	
	36	10	7,290	94,770	
56	47	11	3,610	46,930	49,000
	43	7	5,290	68,770	
	38	2	7,840	101,920	
58	50	4	3,610	46,930	53,400
	44	11	5,760	74,880	
	39	6	8,410	109,330	
60	51	8	4,000	52,000	58,300
	46	3	6,250	81,250	
	40	10	9,000	117,000	

* Length = "laying length" for bell-and-spigot pipe; and face to face of end flanges for flanged pipe.

In selecting a meter tube, adopt one whose maximum capacity is 50 to 100 per cent. above estimated average flow in the pipe line. If one tube does not have enough range in capacity, place two tubes in parallel in connection with one instrument. Ends may be flange, bell or spigot. Unless otherwise specified, flange ends will be made "Master Fitters Standard" for working pressures up to 125 lb. per sq. in. and "Manufacturers Standard" for pressures from 125 to 250 lb. To secure prompt delivery, adopt one of these standards.[27]

Large Venturi meters in Catskill aqueduct are fully described in *E. N.*, Apr. 16, 1914, p. 856.

Simplex water meter* consists of a register, which may be used with a Venturi tube, a Pitot tube, any form of weir, or any form of flume. The register usually indicates, records, and totalizes, but any one of these functions may be omitted when specified. The register transforms either differential heads or varying heads into flows by means of a float and counterbalancing mechanism. It has no theoretical limitations and therefore is capable of measuring the greatest range of flows and the minimum velocities.

DISK, VELOCITY, AND OTHER METERS

Qualifications of a good meter: (*a*) reasonable accuracy at small and large flows, and under all variations of pressure; (*b*) simplicity of mechanism; (*c*) operation by low head, and with but little friction loss†; (*d*) quietness; (*e*)

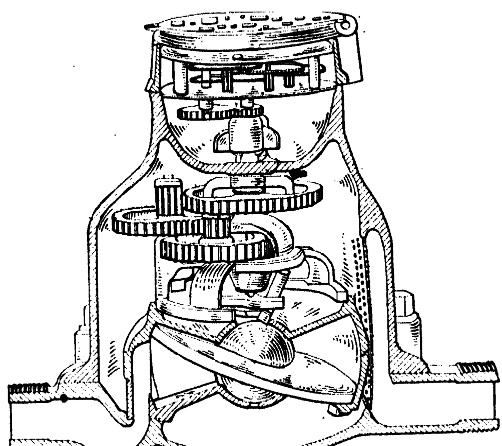

Fig. 202.—Worthington disk meter.

economy; (*f*) durability; (*g*) frost damage minimized; (*h*) low price. The meters mentioned below are used merely as convenient examples; there are many makes on the market (see p. 468) and much difference of opinion as to their relative merits. Meters should be selected with intelligent regard for the service and conditions for which they are to be used.

Disk meters operate by virtue of the nutations of a wobbling disk, supported on a larger ball-and-socket bearing, as it is displaced by the water. The large bearing assures even wear. Makers emphasize the merits of these meters in respect to (*a*) ease of repairs; (*b*) reduction of damage by freezing; (*c*) adjustment for disk wear. Observations by the St. Louis Water Dept.[6] indicate that they retain an accuracy of 90 per cent. or better during life; and that they are 33 per cent. lower in first cost than the rotary type. Disk meters are the most popular type below 3 in.; Detroit[7] uses them up to 6 in. For proper sizes, see Oversized Meters, p. 465.

* Simplex Valve and Meter Co., Philadelphia.

† Tests by C. M. Saville[8] indicate losses in $\frac{3}{4}$-in. meters from 10 to 20 per cent. higher for 4.5 cu. ft. per min. than stated by makers.

Velocity meters (known also as "current,"* "turbine," and "torrent" meters) operate on the principle that the vanes of the wheel move at the same velocity as the water passing; a double wheel, in some types, provides a balance to minimize bearing friction. Current meters are used chiefly above 2 in. and are claimed to measure large flows with comparatively small loss of head; the heavy-pattern turbine meter is adapted to measuring hot water under heavy pressure. A strainer is always provided to keep out injurious solids. On services of proper sizes, accuracy is well maintained. The first and maintenance costs appear lower than for like sizes of other types. Detroit[7] reports dissatisfaction with this type because (*a*) it clogs easily; (*b*) considerable flow is required to produce registration. Philadelphia substituted[8] compound meters, and a large gain in registration resulted.

Rotary meters operate on the displacement principle, piston revolving in guides. The opportunity for grit to enter lowers the accuracy below other types.

Compound meter is designed to avoid the loss of registration on flows too small for accurate measurement by line-size meter;† it consists essentially of three units, a large meter, a small by-pass meter and a regulating valve, so arranged that, as the flow through the by pass reaches a certain quantity, the pressure loss actuates the regulating valve, which opens and diverts all or part of the flow to the larger meter, which is generally of the current type.

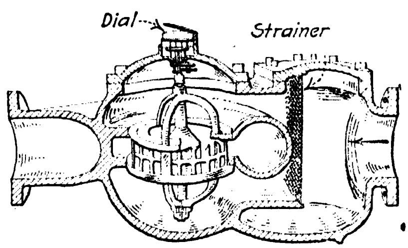

FIG. 203.—Worthington turbine meter.

Experience in Detroit[7] is that meter satisfactorily records small amounts, but the current meter portion is liable to clog. F. B. Nelson[9] suggests that mechanism should be so arranged that large meter does not operate until flows are large enough for accuracy.

Proportional meters‡ effect a saving on large lines by the use of a small disk meter on a by-pass, into which the water is deflected by special "friction rings." The apparatus furnished includes both main pipe and by-pass. The ratio of flow in main to that in by-pass is constant; the meter is geared to record flow in main.

Fire-line meters are required to record both ordinary flows and the large quantities used for fires, and to pass them without appreciable friction losses. Two separate mechanisms record flows, the fire flows being metered by a proportional meter on a by-pass, which leaves the main pipe unobstructed. Hersey Detector meter, Utility meter (Pittsburg Meter Co.), and Trident Protectus meter have approval of National Board of Fire Underwriters.

* The name "current" is objectionable and confusing, because it has so long been used for stream-gaging meters.

† Tests in New York City on 15 meters (3- to 6-in.) showed an aggregate underregistration of 21 per cent. F. B. Nelson.[9]

‡ Hersey Mfg. Co., New York City.

The Associated Factory Mutual Insurance Companies favor meters on fire lines only in exceptional cases.[10]

Meter specifications were adopted by N. E. W. W. A.[11a] and A. W. W. A.[12] in 1923. They cover both disk and velocity meters, and construction details; limits of accuracy for new meters are established. Manufacturers are now prepared to bid on these specifications. Some cities prefer to purchase meters on a performance basis.

Table 132. Lengths of Meters Recommended by Joint Committee[11]
All Dimensions in Inches

Size	Disk	Turbine (current)		Compound	
	Standard length	Minimum length	Maximum length	Minimum length	Maximum length
$\frac{5}{8}$	$7\frac{1}{2}$	....	....	....	....
$\frac{3}{4}$	9	....	....	....	....
1	$10\frac{3}{4}$	....	....	....	....
$1\frac{1}{2}$	$12\frac{5}{8}$	13	$15\frac{1}{4}$	$18\frac{5}{16}$	$18\frac{5}{16}$
2	$15\frac{1}{4}$	$15\frac{1}{4}$	19	$15\frac{1}{4}$	$28\frac{7}{8}$
3	24	20	24	24	$37\frac{1}{8}$
4	29	22	$29\frac{1}{4}$	29	$39\frac{1}{4}$
6	$36\frac{1}{2}$	24	$36\frac{3}{4}$	36	$50\frac{3}{4}$
8	....	$26\frac{3}{4}$	$48\frac{3}{4}$	42	$61\frac{3}{4}$
10	....	30	60	$63\frac{1}{2}$	$72\frac{3}{4}$
12	....	36	70	$64\frac{1}{2}$	77

Accuracy of registration is dependent on the quantities flowing, the pressure, and the wear of the bearings and the operating mechanisms. Chemicals natural to the water or residual from purification processes often affect the accuracy. At Grand Rapids,[13] lime treatment of the water resulted in 10 per cent. of meters being limed, and requiring cleansing. Electrolysis in the gear trains at St. Louis was mitgiated by use of same alloy for all parts.[14] Accuracy can be maintained only by perpetual testing, maintenance, and replacements. Two examples of practice in replacing meters are given in Table 133. Huy[31] increased revenues 50 per cent. by repairing large meters and repairing or replacing small ones.

Table 133. Replacement of Defective Meters[16b]

Size of meters, in.	Maximum registration allowed before meter is replaced, cu. ft.		Maximum time in service before meter is replaced, New Jersey Board, years
	Hackensack Water Co.	New Jersey Board of Public Utility Commissioners	
$\frac{1}{2}$	100,000	750,000	10
$\frac{3}{4}$	250,000	1,000,000	8
1	500,000	2,000,000	6
2	2,000,000	No quantity given	4

Oversized meters should be debarred by regulations; they underregister, resulting in loss of revenue. The Passaic Water Co.[16a] has adopted the following limits: up to 6 families, $\frac{5}{8}$-in. meter; 6 to 12 families, $\frac{3}{4}$-in. meter; 12 to 18 families, 1-in.; 18 to 25 or 30 families, $1\frac{1}{2}$-in. Sizes for industrial lines are based on investigation. Invariable rule is to use smaller size than the service pipe. Pressure losses should be controlled by the service pipe; not by the meter size.

Table 134. Standards of Meter Size in San Francisco[18]

Size, in.	Families	Estimated monthly use, cu. ft.
$\frac{5}{8}$	1–6	4,000
$\frac{3}{4}$	7–10	7,000
1	11–18	12,000
$1\frac{1}{2}$	19–26	24,000
2	27–50	40,000

In Detroit, 74 per cent. of the 154,360 meters in service, July 1, 1922, were $\frac{5}{8}$-in.; 20 per cent., $\frac{3}{4}$-in.; 3 per cent., 1-in.; and the others graded up to 6-in.[7] Of 547 waterworks using meters on residence services, Public Works[19] found that 495 normally use $\frac{5}{8}$-in.; 20 use $\frac{1}{2}$-in.; and 32 allow no size smaller than $\frac{3}{4}$-in.

Testing of meters after service is an important function of operating department. Shop testing consists in passing through the meter a quantity calibrated by weight or volume in a tank. Special piping provided by makers facilitates rapid insertion of meters. Testing machines are made by many meter companies (see p. 468), and Ford Meter Box Co., Wabash, Ind., and H. W. Clark Co., Mattoon, Ill. Small meters should be taken to shop, but large meters are more economically tested in place by portable test meters.

Results of Tests. Of 6400 meters tested in Detroit shop 63 per cent. were found correct; 11 per cent. stopped; 12 per cent. overregistered, and 14 per cent. underregistered from 4 to 25 per cent., averaging 8 per cent. During 1922, 47,345 meters (30 per cent.) were tested; 3980 were frozen; and 5307 (2 per cent.) injured by hot water.* See annual reports, 1922, 1923.

Otto Poetsch[28] tested $\frac{5}{8}$-in. meters, Milwaukee, 1911, none of which had been repaired within 5 years; 3431 meters of piston type gave average slip of 3.15 per cent.; 1955 disk meters gave slip of 0.6 per cent. Piston meters were 15 to 23 years old; disk meters 10 years. On basis of these tests, average slip for city was placed at 1 per cent.

In tests at East Orange, N. J.,[29] July 29, 1908, to June 22, 1909, two Trident and two Empire meters, out of 18 meters, representing six makes, ran constantly under full pressure for $10\frac{1}{2}$ months, with out failure and without serious drop in accuracy. Meters were $\frac{5}{8}$-in. size. Tridents passed 1,857,000 and 1,730,000 cu. ft. respectively, equivalent to 13,927,500 and 12,975,000 gal.; Empires 1,289,000 and 1,273,000 cu. ft., equivalent to 9,667,500 and 9,547,500 gal. Differences in quantities were due to varying capacities of the different machines, accuracy being high in all four. When it is considered that the smallest of above quantities is equivalent to the usual supply of a family of 10 for about 48 years, or a family of 5 for 95 years, the capacity and accuracy of these meters may be appreciated.†

Tests in Des Moines, Iowa, by J. A. Cole, 1897, on 1064 meters in service from 1 to 15 years, showed an average loss of registration of only $1\frac{1}{2}$ per cent.[30]

Less favorable results are frequent; 4000 meters tested by one company in two years showed an accuracy varying from 4 to 90 per cent.

Meter Slippage. Considerable water flows through meters without being registered, varying with the size and depending upon the mechanical condition of the meters. An average slippage of 5 per cent. of the total water distributed is accepted by authorities on meters and confirmed by experience in Detroit.[32]

* Degree of heat not ascertained; makes repaired: Gamon, Lambert, Nash, Thompson, Worthington.

† The Trident is a disk meter, and the Empire of the oscillating-piston type.

Meter operating costs at Terre Haute[20] for 1921 may be taken as typical: Cost of reading 7700 meters averaged 3 cts.; clerical work (billing, receipting, etc.), 85 cts. per customer per year. Cost of repairs, tests, and maintenance averaged 29.0 cts. per meter in service. Inspections, investigations, turning water on and off, removing and setting meters, averaged 56 cts. per meter.

Setting of meters affects costs of reading, maintaining, and testing. Many cities now require a curb setting on new installations wherever conditions permit.

Time required to set $\frac{5}{8}$-in. to 1-in. meters may be taken at 1.75 man-hr.; carting also takes 1.75 man-hr.[17]

Meter boxes may be obtained from makers of cast-iron pipe, Ford Meter Box Co., H. W. Clark (Mattoon, Ill.), S. E. T. Valve & Hydrant Co. (New York), or may be built of masonry in the field. Circular concrete boxes proved cheaper than square in St. Louis.[21] Requisites are frostproofness, space for removing meter, and dial within 18 in. of top. Outside meter boxes cannot always be frostproofed. The severe winter of 1921–1922 in New England "convinced a good many that the small outside meter box is not serviceable in this climate."[22]

Benefits of metering,* according to Committee of Engineers reporting on Chicago, are premised on following conditions which are general in application: (*a*) Minimum rates must be established, which all customers are obliged to pay, whether the water represented is used or not. (*b*) Charges above the minimum will be calculated upon quantity metered. (*c*) The water department will furnish, set, and maintain meters as a municipal expense. (*d*) Meters should be applied indiscriminately to all services, so that they will be regarded as a guide to careful use, rather than as a penalty for waste of water. (*e*) Metering should not be regarded as a means of increasing water revenues, but as a means of postponing and decreasing future investments in mains,† pumping stations, etc., and of increasing pressures. If water revenues become the prime consideration, the expense of metering will become a burden, and defeat its merits.[23]

Results of Metering in Boston.[24] J. A. McMurry states:

> Facts accumulated year after year force one to the conclusion that a metered city will not only keep the consumption within reasonable limits but will also receive sufficient income to meet department expenses. Water takers are getting satisfactory bills (and in thousands of cases bills lower than under the annual-rate plan) and the department is receiving more than sufficient to pay its bills, with quite a large surplus.

In 1908, daily consumption was 98,300,000 gal. and per capita 158 gal. In 1916, when the city was a little over 50 per cent. metered, daily consumption was 80,388,000 gal. and per capita 105 gal., a decrease of 18,000,000 gal. a day between 1908 and 1916. In 1921, daily consumption was 85,609,000 gal. a day, 112 gal. per capita, a difference of 13,000,000 between 1908 and 1921. In forecasting reduction in consumption, it is well to remember that American cities are showing a progressively higher per-capita rate as population increases. J. Waldo Smith[25] warns that estimates should consider this fact, which holds true whether or not meters are installed.

* See also p. 426.

† Clifton, Arizona installed meters and reduced the demands 50 per cent., at a cost of 12 per cent. of the estimated cost for a new supply.[33]

Table 135. Effect of Meters on Water Consumption[26]*

Per cent. of services metered	Cities 5,000 to 25,000 population		Cities 25,000 to 100,000 population		Cities 100,000 population* and over	
	Number of cities	Gallons per capita per day	Number of cities	Gallons per capita per day	Number of cities	Gallons per capita per day
0 to 10	60	161	16	198	3	218
10 to 20	20	199	3	144	3	149
20 to 30	17	128	2	208	3	154
30 to 40	15	108	4	116	3	170
40 to 50	8	88	5	105	3	160
50 to 60	18	141	5	165	0	0
60 to 70	18	120	4	91	0	0
70 to 80	16	140	10	84	4	97
80 to 90	31	79	9	107	10	127
90 to 100	145	91	53	94	22	97
Total.....	348	....	111	....	51	....

* See also Hill in *Am. City*, Vol. 29, 1923, pp. 8 and 158.

Regulations and Rates. An epitome of current (1919) practice has been put out in mimeograph form by State Bureau of Municipal Information, of the New York State Conference of Mayors and other City Officials (W. P. Capes, Director, Albany, N. Y.). For experience of American cities, see digest of questionnaire, *Ibid.*, August 1917 (see also p. 832).

Table 135A. An Identifying Directory of Makers of Meters

American—Buffalo Meter Co., Buffalo, N. Y.
Arctic—Pittsburgh Meter Co., East Pittsburgh, Pa.
Badger—Badger Meter Co., New York City
Columbia—Union Meter Co., Worcester, Mass.
Crest—Neptune Meter Co., New York City
Crown—National Meter Co., New York City
Detector—Hersey Mfg. Co., South Boston, Mass.
Empire—National Meter Co.
Eureka—Pittsburgh Meter Co.
Federal—Federal Meter Co., New York City
Gem—National Meter Co.
Hersey—Hersey Mfg. Co.
Keystone—Pittsburgh Meter Co.
King—Union Meter Co.
Lambert—Thomson Meter Co., New York City
Nash—National Meter Co.
Niagara—Buffalo Meter Co.
Nilo—Union Meter Co.
Premier—National Meter Co.
Proportional—Hersey Mfg. Co.
Protectus—Neptune Meter Co.
Thomson—Thomson Meter Co.
Torrent—Hersey Mfg. Co.
Trident—Neptune Meter Co.
Union—Union Water Meter Co.
Watch Dog—Gamon Meter Co., Newark, N. J.
Worthington—Worthington Pump & Machinery Co., New York City

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CHAPTER 22

PUMPS, PUMPING STATIONS, AND EQUIPMENT

Service Required. The most exacting use of small and medium-sized pumping equipment is fire service; adequate supply at proper pressure,* with maximum number of streams, requires (1) several small pumps of aggregate capacity considerably above domestic demand, which necessitates some idle equipment; or (2) large pumps normally operated at small load factors, with consequent low efficiencies; or (3) a reservoir to supply fire needs; or (4) a combination of pumps and reservoirs. The three principal considerations[1] in designing a pumping station are reliability, adequacy, and economy. Reliability can be obtained by best type of equipment in duplicate; adequacy, by using liberal sizes; economy involves life and depreciation, first cost, stand-by charges, interest, and operating costs.

National Board of Fire Underwriters has issued elaborate specifications for fire-fighting requirements: dual power supply for reliability; pumps housed in fireproof structures, effectively operated and maintained. Secondary sources of power for electric drive are generally Diesel or gasoline engines. In steam stations there is not the same necessity for dual power as where motors or internal-combustion engines are prime movers. Pumping capacity sufficient to meet maximum domestic demand combined with fire flow at required pressure, when the two largest pumps are out of service, is recommended. On small systems for fire demands the pump capacity may be three to five times the domestic requirements. Johnson[2] holds that in a small system it is not feasible to have sufficient pump capacity for direct fire service, and that dependence for fire service must rest on a reservoir or on factory pumps. Compilation by Johnson of towns in Massachusetts with less than 6000 population indicates that pump capacity is from 2 to 25 times the average daily consumption. See also epitome of practice in 66 cities, *E. C.*, May 10, 1918, p. 453.

Relation to Distribution Reservoir.† Many early pumping systems had no distribution reservoir; pumps operated at different speeds, controlled by automatic regulators on steam throttle. This was known as the "direct pumping system." Modern electrically operated pumps on a direct pumping system can be controlled and put into service successively by electrical controls, but close conformity to demands is not attainable. There is lack of economy in operating a direct pumping system; with a reservoir, a reciprocating pump operates at constant speed against reservoir pressure at higher efficiency, or electrically operated pumps may be more closely controlled by float controllers in the reservoirs. Direct pumping causes higher water-hammer effects than where there is a standpipe on the line. A standpipe

* See p. 399.

†See also p. 533.

5 ft. in diam. and 224 ft. high on the Toledo system was torn down in 1916 after trials had shown that both the main and the pumps would not be badly affected by direct pumping.[90]

On small systems, pumping to a reservoir or standpipe reduces cost for attendance, power, and stand-by pumps for fire service. Where steam pumps are employed, pumping may be restricted to day time, and fires banked at night, cutting down fuel costs. With electrically driven pumps (common now on smaller systems) large economy results from limiting pumping to night time, when power demands on electric utility corporations are low, and favorable rates may be secured (see p. 525). Investigations of pumping economics must consider the number of hours a day the plant is to be operated. With large reservoir capacity 8-hr. pumping will usually suffice. With an ample standpipe and a small demand at night, many stations operate with two shifts, totaling 16 or 18 hr. Other plants pumping against direct pressure have to operate a full 24-hr. day.

Booster Service. Topographical conditions are often such that pressures adequate for low-lying portions of municipalities result in deficient pressures in elevated areas. To meet this condition booster pumps are installed on distribution system directly, or are connected to small suction wells, to pump to higher levels at adequate pressure. Many of these booster pumps are electrically operated, and started and stopped by electrical controls within prescribed pressure limits (see p. 525).

Pumping vs. Gravity. A gravity supply requires development of a watershed (see Chap. 6) and commonly a long conduit to the distribution system. Wherever possible, the reservoir is at sufficient elevation to afford adequate pressures at all times. Otherwise booster pumps supply high-service areas. Considerations of economy must dictate the proportion of work imposed on boosters. The first cost of a gravity system is often high, due to real estate for reservoir site and conduit, and the fact that reservoir structures and conduits are proportioned to meet demands for many decades. On other hand, operating costs are small outside of interest charges and depreciation of conduit, or reservoir; but these may be large. Reliability is generally greater than with a direct-pumping system. The direct-pumping system commonly involves a short conduit to the distribution system, and small first cost for pumping station and conduit; operating cost is, however, high, comprising chiefly power, depreciation of pumping equipment and buildings, repairs, and attendance.

Types of Pumps.* Pumping machinery moves water to a new position (1) by displacing its volume, as in displacement pumps; (2) by *continual* application of mechanical energy, as in centrifugal pumps; or (3) by impulse, as in the ram. Displacement pumps, of which the ordinary reciprocating pump is the best-known type, operate by creating a space in the pump cylinder by displacement of a volume which is forced into the discharge pipe. The motive energy of the displacement pump results from direct pressure trans-

* Great varieties of pumping units are offered by a large number of makers; many of them are quite satisfactory for usual requirements. Much additional useful information may be had from the publications of the best manufacturers. Reference is also made to "Centrifugal Pumping Machinery," C. G. De Laval (McGraw-Hill Book Company, Inc., 1912); "Centrifugal Pumps," Loewenstein and Crissey (Van Nostrand, 1911); "Centrifugal Pumps," R. L. Daugherty (McGraw-Hill Book Company, Inc., 1915); "Centrifugal Pumps," J. W. Cameron (Scott, Greenwood, London, 1921); "Modern Pumping and Hydraulic Machinery," E. Butler, (Griffin, London, 1922).

mitted to the water by the piston. In centrifugal pumps the energy results from the velocity of the impeller, and in jet pumps from the moving jet. The air-lift pump is discussed in Chap. 10. A centrifugal pump driven by a steam turbine is known as a turbo-centrifugal. Steam engines incorporated with pumps in single machines have been known as pumping engines but this term is now extended to pumps combined with gas or oil-engine-drive. Pumps requiring a separate prime mover are *power pumps*.

Sizes of Units. Pump sizes for small communities are dictated by requirements of National Board of Fire Underwriters, which, unless there is a reservoir, involve a large amount of idle equipment, operated only for fires. A common error in design of pumping stations is to divide the maximum total pumping capacity required into units of equal capacity. No waterworks pump is operated at full capacity all the time. To meet conditions most economically, install pumps of several sizes, the smallest pump being capable of meeting domestic demands at night. Usually, division into different sizes will allow each unit to be operated at full capacity. By intelligent division, the station may be worked at maximum efficiency for a large proportion of the day.[3] Recent practice favors the use of electrically driven auxiliaries (see p. 522) to take peak loads, and to afford stand-by service.

Pump Selection.[4] Points to be considered: (1) The pump should be of an established make, so that its performance under varying conditions is known, and parts readily replaceable. Only the largest municipalities can afford to design their pumps; others use commercial designs. (2) First cost, only where two pumps fulfil all other qualifications. (3) Dependability, particularly where there is no reserve storage; it can be ascertained only by inspecting records of similar plants. (4) Life of plant, which affects depreciation reserve. (5) Size of floor space available. (6) Head. (7) Plant efficiency:* ratio of power output of pump to power input of plant; this is commonly the determining factor in selecting pumping machinery. (8) Commercial efficiency: the total cost per unit volume of water pumped during life of plant should also govern, although difficult to evaluate; it should embrace first cost, insurance, taxes, depreciation, operation, upkeep, repairs, replacements, and all losses between power received and water delivered. (9) Flexibility, which enables the equipment to meet varying operating conditions; flexibility of all pumps is limited, but the reciprocating steam pump and the rotary pump have greatest. (10) Priming requirements.

Power Calculations.† Pumping head should include (*a*) static head, low water to high water, (*b*) friction in pipes, (*c*) velocity head. In computing (*b*), it should be recognized that throwing an additional pump into service adds to the quantity, and may increase friction losses materially.

$$\text{Brake hp.} = \frac{\text{G.p.m.} \times \text{Head, in ft.}}{4000 \times \text{Pump efficiency}} \quad \ldots \text{(error, plus 1.1 per cent.)} \qquad (1)$$

Millon ft.-lb. duty per million B.t.u. =

$$\frac{1{,}980{,}000}{\text{lb. steam per hr. per brake hp.} \times \text{B.t.u. per lb.}} \qquad (2)$$

* Officials are often misled as to the efficiency and operating costs by official tests under full load, whereas the efficiency falls off under normal operating conditions of partial loads and intermittent operation.

† See also p. 491.

$$\text{Million ft.-lb. duty per 1000 lb. steam} = \frac{1980 \times \text{pump eff.}}{\text{Lb. steam per brake hp.-hr.}} \tag{3}$$

$$\text{Pounds of steam per hr. per i.h.p.} = \frac{1{,}980{,}000}{\text{Duty in ft.-lb. per 1000 lb. of steam}} \tag{4}$$

$$1 \text{ B.h.p.} = 746 \text{ watts} \tag{5}$$

$$\text{Kw.-hr. per month} = \frac{\text{Mgd.} \times 94.2 \times \text{Head, in ft.}}{\text{Combined efficiency of pump plus motor}} \tag{6}$$

The full-load efficiency for power pump varies from 50 per cent. for the small-size centrifugals to over 80 per cent. for the high-duty reciprocating pumps, and large centrifugals.

Altitude, Pressure, and Suction. The height that a pump will lift water by suction depends on atmospheric pressure, which decreases as altitude increases, roughly 0.5 lb. per sq. in. for every 1000 ft. of ascent. At altitude 1500 ft., the pressure is 0.75 lb. less than at sea level, or roughly 14 lb. per sq. in. At sea level 1 atmosphere has a pressure of 14.696 lb. per sq. in., or a head of

Table 136. Pressures Corresponding to Barometer Heights

Barometer, in.	Pressure per sq. in., lbs.	Barometer, in.	Pressure per sq. in., lbs.
28	13.74	29¾	14.60
28¼	13.86	30	14.72
28½	13.98	30¼	14.84
28¾	14.11	30½	14.96
29	14.23	30¾	15.09
29¼	14.35	31	15.21
29½	14.47		

33.94 ft. of water, or 29.922 in. of mercury. Moisture in air makes a difference in pressure. The readings of a barometer take care of both altitude and moisture corrections. Theoretical horsepower required is given on p. 473. To find theoretical lift of pump in ft., multiply pressure* per sq. in. from barometer reading by 2.3. Of course the pump will not lift this height, as it cannot produce a perfect vacuum,† and even if it could, there must be enough difference between pressure in pump chamber and atmosphere not only to sustain the height of column, but to overcome its friction.

* For table of pressures, see p. 818.
† For table of vacuums, see p. 817.

Table 137. Theoretical Horsepower Required to Raise Water to Different Heights

Feet		5	10	15	20	25	30	35	40	45	50
Lbs. per sq. in.		2.2	4.3	6.5	8.7	10.8	13.0	15.2	17.3	19.5	21.7
Gal. per min.	Mgd.										
5	0.007	0.006	0.012	0.019	0.025	0.031	0.037	0.044	0.05	0.06	0.06
10	0.014	0.012	0.025	0.037	0.050	0.062	0.075	0.078	0.10	0.11	0.12
15	0.022	0.019	0.037	0.056	0.075	0.094	0.112	0.131	0.15	0.17	0.19
20	0.029	0.025	0.050	0.075	0.100	0.125	0.150	0.175	0.20	0.22	0.25
25	0.036	0.031	0.062	0.093	0.125	0.156	0.187	0.219	0.25	0.28	0.31
30	0.043	0.037	0.075	0.112	0.150	0.187	0.225	0.262	0.30	0.34	0.37
35	0.050	0.043	0.087	0.131	0.175	0.219	0.262	0.306	0.35	0.39	0.44
40	0.058	0.050	0.100	0.150	0.250	0.300	0.300	0.350	0.40	0.45	0.50
45	0.065	0.056	0.112	0.168	0.225	0.281	0.337	0.304	0.45	0.51	0.56
50	0.072	0.062	0.125	0.187	0.250	0.312	0.375	0.437	0.50	0.56	0.62
60	0.086	0.075	0.150	0.225	0.300	0.375	0.450	0.525	0.60	0.67	0.75
75	0.108	0.093	0.187	0.281	0.375	0.469	0.562	0.656	0.75	0.84	0.94
90	0.129	0.112	0.225	0.337	0.450	0.562	0.675	0.787	0.90	1.01	1.12
100	0.144	0.125	0.250	0.375	0.500	0.625	0.750	0.875	1.00	1.12	1.25
125	0.180	0.156	0.312	0.469	0.625	0.781	0.937	1.094	1.25	1.41	1.56
150	0.216	0.187	0.375	0.562	0.750	0.937	1.125	1.312	1.50	1.69	1.87
175	0.252	0.219	0.437	0.656	0.875	1.094	1.312	1.531	1.75	1.97	2.19
200	0.288	0.250	0.500	0.750	1.000	1.250	1.500	1.750	2.00	2.25	2.50
250	0.360	0.312	0.625	0.937	1.250	1.562	1.875	2.187	2.50	2.81	3.12
300	0.432	0.375	0.750	1.125	1.500	1.875	2.250	2.625	3.00	3.37	3.75
350	0.504	0.437	0.875	1.312	1.750	2.187	2.625	3.062	3.50	3.94	4.37
400	0.576	0.500	1.000	1.500	2.000	2.500	3.000	3.500	4.00	4.50	5.00
500	0.720	0.625	1.250	1.875	2.500	2.125	3.750	4.375	5.00	5.62	6.25

Feet	60	75	90	100	125	150	175	200	250	300	350	400
Lbs. per sq. in.	26	33	39	43	54	65	76	87	108	130	152	173
Gal. per min.												
5	0.07	0.09	0.11	0.12	0.16	0.19	0.22	0.25	0.31	0.37	0.44	0.5
10	0.15	0.19	0.22	0.25	0.31	0.37	0.44	0.50	0.62	0.75	0.87	1.0
15	0.22	0.28	0.34	0.37	0.47	0.56	0.66	0.75	0.94	1.12	1.31	1.5
20	0.30	0.37	0.45	0.50	0.62	0.75	0.87	1.00	1.25	1.50	1.75	2.0
25	0.37	0.47	0.56	0.62	0.78	0.94	1.09	1.25	1.56	1.87	2.19	2.5
30	0.45	0.56	0.67	0.75	0.94	1.12	1.31	1.50	1.87	2.25	2.62	3.0
35	0.52	0.66	0.79	0.87	1.08	1.31	1.53	1.75	2.19	2.62	3.06	3.5
40	0.60	0.75	0.90	1.00	1.25	1.50	1.75	2.00	2.50	3.00	3.50	4.0
45	0.67	0.84	1.01	1.12	1.41	1.69	1.97	2.25	2.81	3.37	3.94	4.5
50	0.75	0.94	1.12	1.25	1.56	1.87	2.19	2.50	3.12	3.75	4.37	5.0
60	0.90	1.12	1.35	1.50	1.87	2.25	2.62	3.00	3.75	4.50	5.25	6.0
75	1.12	1.40	1.69	1.87	2.34	2.81	3.28	3.75	4.69	5.62	6.56	7.5
90	1.35	1.68	2.02	2.25	2.81	3.37	3.94	4.50	5.62	6.75	7.87	9.0
100	1.50	1.87	2.25	2.50	3.12	3.75	4.37	5.00	6.25	7.50	8.75	10.0
125	1.87	2.34	2.81	3.12	3.91	4.69	5.47	6.25	7.81	9.37	10.94	12.5
150	2.25	2.81	3.37	3.75	4.69	5.62	6.56	7.50	9.37	11.25	13.12	15.0
175	2.62	3.28	3.94	4.37	5.47	6.56	7.66	8.75	10.94	13.12	15.31	17.5
200	3.00	3.75	4.50	5.00	6.25	7.50	8.75	10.00	12.50	15.00	17.50	20.0
250	3.75	4.69	5.62	6.25	7.81	9.37	10.94	12.50	15.72	18.75	21.87	25.0
300	4.50	5.62	6.75	7.50	9.37	11.25	13.12	15.00	18.75	22.50	26.25	30.0
350	5.25	6.56	7.87	8.75	10.94	13.12	15.31	17.50	21.87	26.25	30.62	35.0
400	6.00	7.50	9.00	10.00	12.50	15.00	17.50	20.00	25.00	30.00	35.00	40.0
500	7.50	9.37	11.25	12.50	15.62	18.75	21.87	25.00	31.25	37.50	43.75	50.0

Pumping costs* depend on first cost of plant, on type of pumps, quantity, head, price of fuel† and supplies, and charges for electric energy. Pumping costs are commonly expressed as the cost of raising 1,000,000 gal. 1 ft. high. Rising prices of coal since the war have made fuel a larger part of the total operating cost than previous to 1918. The variation in basic costs for pump-

* See statistics for 66 U. S. cities in *E. C.*, May 10, 1918, p. 453.

† See p. 513.

ing over a period of years for Chicago are offered as a guide in applying past costs to recent conditions.

Table 138. Pumping Costs at Chicago. Effect of Price Changes[93]

Year	Aver. head pumped against, feet	Aver. pumpage, mgd.	Coal used, 1000 tons	Aver. cost of coal per ton	Aver. cost 1 mgd. 1 ft. high, cents
1912	115.8	551.3	166.6	$2.38	3.90
1913	120.2	575.9	170.4	2.21	3.58
1914	120.7	613.3	182.5	2.14	3.53
1915*	115.4	606.7	193.8	2.08	3.69
1916	113.3	628.0	193.6	2.00	3.58
1917	110.3	641.5	201.4	3.20	4.92
1918	105.2	657.9	201.0	4.17	6.00
1919	108.2	714.5	202.2	4.13	6.20
1920	108.8	760.1	189.0	5.74	7.40
1921	113.9	775.1	173.1	6.24	8.70
1922	115.2	785.5	147.4	6.86	7.70
1923	118.0	788.5	151.0	4.54	6.70

* Figures previous to 1915 lack the accuracy of later figures.

RECIPROCATING PUMPING ENGINES*

Types. Reciprocating pumps may have one or more water cylinders; the number of cylinders determines the name, duplex, triplex, or quadruplex. Steam end may be simple, compound, or triple expansion. If the plunger does not come in contact with the cylinder wall, the pump is known as a plunger pump. Where the pump plunger is driven from the cross-head of a steam engine, the machine is called a crank-and-flywheel pumping engine.

The single pump[4] (double cylinder) has a pulsating discharge; an air chamber† must be provided to absorb the shock which would cause water-hammer in pipes. This type is simple and reliable, and has low steam consumption. It must be started by hand.

The duplex pump has a less pulsating discharge, and the air chamber is frequently unnecessary. It can be used with automatic starters. This type is largely used for steam fire pumps; National Board of Fire Underwriters has issued regulations for manufacture and installation.

Triplex pumps‡[6] (single-acting plunger pumps) are common in small plants. Three cranks set 120° apart give uniform flow, fairly free from shock. Air chambers are, however, generally desirable. Plungers are easily inspected and renewal of packing is easy. Triplex pumps should not be used in residential districts, due to excessive noise in gearing. They readily meet demands for fire service, as they can be operated with good efficiency against double pressure. Triplex pumps with long rods can pump from pits. A large triplex pump under favorable conditions may have an efficiency of nearly 80 per cent.[6]

* Reader is referred to "Direct-acting Steam Pumps," by F. F. Nickel (McGraw-Hill Book Company, Inc., 1915); "Pumping Machinery," by Greene (Wiley, 1919), "Modern Pumping and Hydraulic Machinery," by Butler (Griffin, 1922); new pump standards promulgated in 1922 by the manufacturers (Hydraulic Society, New York).

† For design, see 1913 *Report*, Chicago Public Works Dept.

‡ Makers include Aldrich Pump Co., Deane Steam Pump Co., Deming Co., Gould Pump Co., Novo Engine Co., Rumsey Pump Co., Worthington Pump & Machinery Co.

Triplex pumps may be operated by steam, gas, or electricity; may be belt or chain driven, or directly connected to motor, engine, or water-wheel through reduction gearing. Small sizes when intermittently used should not be operated by steam on account of heat losses when idle. Motors and gas engines require reduction gearing.

Cross-compound. Although first cost of a compound pump is greater than that of a simple pump, it is offset by smaller boiler capacity and 20 to 30 per cent. fuel saving. For service against reservoir or standpipe head, or where but slight increase of pressure is necessary for fire service, a cross-compound pump is simpler, and, as a rule, less expensive than the three-cylinder type.[3] The cross-compound flywheel is generally considered most economical type for 3- to 5-mgd. plants,[7] but may be questioned on basis of duty per dollar of annual charges. First cost is much less than that of vertical triple-expansion, crank-and-flywheel type. A duty of 125,000,000 to 150,000,000 ft.-lb. can be obtained.

Triplex Compound Pump.[3] Essential feature is the distribution of the low-pressure load between two cylinders and frames, instead of concentrating on one, as in the cross-compound. Where the latter is to be used on a steady load, cylinders can be so proportioned as to throw on each of the high- and low-pressure frames approximately equal loads. Where there is a wide variation between domestic and fire pressure, advantage of three-cylinder arrangement is marked; live steam may be admitted to the receiver without danger, since the load, which in a cross-compound would come on one frame, is distributed on two, each as strong as the high-pressure frame, so that no matter how hurriedly or unskilfully the machine is handled while pushing the service up to fire pressure, chances of damage at this critical time are reduced a half. In a duplex double-acting flywheel machine the minimum flow is 54 per cent. of the maximum, while in a three-cylinder machine the minimum is approximately 74 per cent. of the maximum.

Triple-expansion Flywheel Pumping Engine. Vertical engines of this type have until recently been preeminent from an economic and mechanical standpoint for large installations. The early cut-off in the steam cylinders allows large utilization of the expansion of the steam, with consequent saving in steam consumption. Under favorable steam and water conditions, a duty exceeding 200,000,000 ft.-lb. per 1000 lb. of steam has been obtained. These engines have been reliable and maintenance cost low, but they require expensive buildings and foundations, and the investment is high. Cost is prohibitive in small plants, owing to high fixed charges, which will annually exceed $700 per mgd. capacity.[1]

Quadruple-expansion pumping engines,[3] resulting either in a tandem arrangement, or in abandonment of three-plunger designs so favorable to uniform hydraulic effects, are not suitable for waterworks service.

Uniflow pumping engine[8] takes advantage of steam flow into and through the steam cylinder in one direction only. The heat energy is utilized in the cylinder during the period of its admission, expansion, and flow in one direction, the expanded steam being exhausted through ports remote from point of admission, so that the comparatively cold steam has least effect on the hot end of the cylinder. There is a heat gain over counterflow cylinders in the

elimination of initial condensation. Some American manufacturers have guaranteed as low as 10 lb. of steam per i.hp. per hr., and European engines have tested well under 9. With a single cylinder, substantially the economy of the best compound or triple-expansion engine is obtained.

Merits of Reciprocating Steam Pumps.[4] *Advantages:* (1) low first cost, (2) durability, (3) flexibility, (4) ability to operate against a high head, (5) operation easily understood, (6) high efficiency usually obtainable.*

Disadvantages: (1) frequent adjustment, (2) inability to pump sandy water, (3) weight and size, (4) lack of simplicity, (5) priming usually necessary unless piston packings are well maintained, (6) necessity for vacuum and air chambers on the inlet and discharge pipes to equalize flow and prevent shocks, (7) cannot be directly connected to motors, (8) can rarely be directly connected to internal-combustion engines.

Merits of Piston or Plunger Pump. *Advantages:* (1) durability, (2) operation against high head, (3) uniform flow for multi-cylinder, (4) high efficiency, (5) good suction. Pumps of piston type, owing to facility with which packing can be renewed, and to smaller clearance spaces in pump cylinders, are particularly efficient for lifting water by suction, where it is impossible to prime the suction piping before starting the pump.[3]

Disadvantages: (1) high first cost, (2) limited flexibility, (3) great weight and size, (4) operation at slow speed, (5) priming necessary, (6) sandy water not pumped successfully. They are considered by Vanleer[4] the most efficient type for heads over 300 ft.

Reciprocating vs. Centrifugal Pumps. Reciprocating pumps have a nearly uniform efficiency over a large range, whereas efficiency of centrifugal pumps is affected by any change in head or speed. This makes for greater flexibility in operation of reciprocating pumps. Repairs and renewals do not require expert labor. For common suction conditions a properly packed pump[83] requires neither priming device nor foot valve. Reciprocating outfits cost more, occupy more floor space, require heavier foundations, require more attendance, make more noise, require air chambers on discharge lines to avoid water-hammer, and cannot be driven by high-speed motors or engines except through reduction gearing, with consequent loss in efficiency.

Power Calculations.[3] Dividing water horsepower† by mechanical efficiency gives theoretical indicated horsepower. From Table 139, accepting the usual average mechanical efficiency given in the last column for a pump of capacity given in the first column, the approximate required indicated horsepower can be obtained. Then from Table 151,‡ having determined the type of engine to be installed, and knowing its ordinary duty, get the number of pounds of steam per i.hp. required to be furnished by the boilers. It is usually safe to provide an excess for steam consumption of auxiliaries and losses due to condensation in steam pipes, of about 10 to 15 per cent. of the steam required for main pumping units.

* Efficiency varies with length of stroke from 40 per cent. for 3 or 4 in., to 80 per cent. for 24 in.
† See p. 473. ‡ p. 506.

Table 139. Mechanical Efficiency of Pumping Engines; Indicated Horsepower in Steam Cylinders[3]

Capacity mgd. (24 hours)	Total water load against plunger including suction, pounds pressure per sq. in. — Indicated horsepower of steam cylinders												Mech.* eff. in per cent. of indicated horsepower
	40	50	60	70	80	90	100	110	120	130	140	150	
0.1	2	3	4	4	5	6	6	7	7	8	9	9	65
0.2	5	6	7	8	10	11	12	13	14	15	17	18	68
0.3	7	9	10	12	14	15	17	19	21	23	24	26	71
0.4	9	11	13	16	18	20	22	24	27	29	31	33	73
0.5	11	13	16	19	22	24	27	30	32	35	38	40	75
0.6	13	16	19	22	25	28	32	35	38	41	44	47	77
0.7	15	18	22	25	29	33	36	40	44	47	51	55	78
0.8	16	21	25	29	33	37	41	45	49	53	57	62	79
0.9	18	23	27	32	36	41	46	50	55	59	64	68	80
1.0	20	25	30	35	41	46	51	56	61	66	71	76	80
1.5	30	38	45	53	60	68	75	83	90	98	105	113	81
2.0	40	49	59	69	79	89	99	109	119	128	138	148	82
2.5	49	61	73	85	98	110	122	134	146	159	171	183	83
3.0	58	72	87	101	116	130	145	159	174	188	203	217	84
4.0	76	95	114	133	153	172	191	210	229	248	267	286	85
5.0	94	118	141	165	188	212	236	259	283	306	330	353	86
6.0	112	140	168	196	223	251	279	307	335	363	391	419	87
7.0	130	163	195	228	261	293	326	358	391	424	456	489	87
8.0	147	184	221	258	295	331	368	405	442	479	515	552	88
9.0	166	207	248	290	331	373	414	456	497	538	580	621	88
10.0	182	228	273	319	364	410	455	501	546	592	637	683	89
11.0	200	250	300	350	401	451	501	551	601	651	701	751	89
12.0	216	270	324	378	432	486	540	594	648	702	756	810	90
13.0	234	293	351	410	468	527	585	644	702	761	819	878	90
14.0	248	312	374	436	499	561	623	686	748	810	873	935	91
15.0	267	334	401	467	534	601	668	735	801	868	935	1002	91
16.0	282	352	423	493	564	634	704	775	745	916	886	1057	92
17.0	299	374	449	524	599	674	748	823	898	973	1048	1123	92
18.0	314	392	470	549	627	706	784	862	941	1019	1098	1176	93
20.0	348	436	523	610	697	784	871	958	1045	1132	1220	1307	93
22.0	379	474	569	664	758	853	948	1043	1138	1232	1327	1422	94
25.0	431	539	646	754	862	969	1077	1185	1293	1400	1508	1616	94
30.0	511	640	767	895	1022	1150	1279	1406	1534	1662	1789	1917	95
35.0	597	746	895	1045	1194	1343	1492	1642	1791	1940	2089	2338	95
40.0	675	844	1013	1182	1350	1519	1688	1857	2025	2194	2363	2532	96

* Mechanical efficiency including hydraulic losses and volumetric losses (slip) = Pump horsepower ÷ indicated horsepower of steam cylinders. Pump horsepower = product of plunger displacement, in g.p.m., weight of 1 gal., in lb., and head in ft. divided by 33,000 = product of capacity n m.g.d., 1.547, water load against plunger in lb. per sq. in. and 144, divided by 550.

Best reciprocating pumping plants[9b] involve: Vertical, triple-expansion, crank-and-flywheel pumping engines; long stroke, rotative speed not to exceed 20 r.p.m.; maximum piston travel 200 ft. per min.; modified steam jacketing and reheating; steam pressure at throttle, 175-lb. gage; moderately superheated steam by independent apparatus; smoke-flue reheating; water-tube boilers; mechanical stokers; natural draft at least 0.8 in. of water; feed-water economizers; automatic damper regulators; coal bought on basis of 14,000 heat units per lb.; boiler efficiency of 75 per cent.; coal per i.hp., 1 lb. for large plants, 1.75 lb. for small plants; maintenance of engines, 1.5 per cent. for large plants, 3 per cent. for small plants.

Table 140. Trial Duty Performance of Steam-driven Condensing Pumping Engines[3]

Type	Duty, Million Ft.-lbs. per 1000 Lbs. Dry Steam
Vertical, triple expansion, crank and flywheel (1918)[11a]	201 to 211
Horizontal, cross compound, crank and flywheel (1917)[11a]	150 to 175
Horizontal, duplex, direct-acting, triple	75 to 100
Horizontal, duplex, direct-acting, compound	50 to 70
Turbine-driven, centrifugal pumps[12]	80 to 160
Engine-driven, centrifugal pumps	70 to 110

Many place too much value upon trial engine duty; it is important, but effectiveness of remainder of plant is of equal consequence. Sight must not be lost of fact that cost of coal, which is the item chiefly affected by high engine duties, is, in most cases, but 25 to 40 per cent. of total operating cost and a much lower per cent. of entire cost of pumping.

Table 141. Duty and Consumption of Fuel of Steam Pumping Plants

S = pounds of water per hp.-hour, S_1 = pounds of coal per hp.-hour; E = mechanical efficiency, per cent.; $S_1 = (S \times 100) \div E$; find result in second column. For duty and fuel consumption find values opposite. Fuel is based on raising 100 gals. per min., 100 ft. high for 24 hrs.

Duty in million foot-pounds per 1000 lbs. dry steam	Steam consumption per water-horsepower, lbs. per hr.	Coal in tons per 24 hrs. Evaporation*			Oil in barrels, 42 gals., per 24 hrs. Evaporation†			
		6 to 1	8 to 1	9 to 1	12 to 1	13 to 1	14 to 1	15 to 1
30.3	65.0	0.328	0.246	0.220	0.93	0.86	0.80	0.74
33.0	60.0	0.302	0.227	0.204	0.86	0.79	0.74	0.685
36.0	55.0	0.280	0.208	0.186	0.79	0.73	0.68	0.63
38.7	51.0	0.258	0.193	0.172	0.73	0.67	0.63	0.58
44.5	44.5	0.224	0.167	0.150	0.63	0.60	0.55	0.50
50.0	39.6	0.199	0.149	0.133	0.57	0.52	0.485	0.45
55.5	35.7	0.179	0.134	0.120	0.51	0.47	0.44	0.40
60.5	32.5	0.164	0.122	0.109	0.46	0.43	0.40	0.37
66.5	29.7	0.149	0.112	0.100	0.42	0.39	0.365	0.34
72.0	27.5	0.138	0.104	0.092	0.39	0.36	0.34	0.315
77.5	25.5	0.128	0.096	0.085	0.36	0.325	0.315	0.29
82.5	23.8	0.120	0.090	0.081	0.34	0.315	0.293	0.272
88.0	22.4	0.112	0.085	0.075	0.32	0.295	0.275	0.255
94.0	21.0	0.105	0.080	0.071	0.30	0.275	0.26	0.24
100.0	19.8	0.100	0.075	0.067	0.29	0.26	0.245	0.225
105.0	18.8	0.095	0.072	0.064	0.27	0.25	0.23	0.215
110.0	18.0	0.091	0.068	0.059	0.25	0.235	0.22	0.205

* Ratio of water evaporated to dry coal consumed.
† Ratio of lb. of water evaporated to lb. of oil consumed.

Table 142. Items for and against High-duty* Pumping Plant[9a]

AGAINST HIGH-DUTY:	IN FAVOR OF HIGH-DUTY:
Maintenance account for machinery	Maintenance account for boilers
Interest on machinery	Interest on boilers
Oil, waste, packing, etc.	Sinking fund for boilers
Sinking fund for machinery	Coal account
	Early cut-off in steam cylinders allows large utilization of expansive power of steam.

*Makers of high-duty pumping engines include Allis-Chalmers Mfg. Co.; Hoover, Owens, Rentschler Co., Hamilton, Ohio; Worthington Pump & Machinery Corp., R. D. Wood & Co., Philadelphia.

Proposals for Pumping Engines. Wide-open proposals for engines generally result in large variations in the bids. Exempting the engine contractor from all foundation work, etc. generally results in better work and prices. The data to be furnished bidders consist of (a) static water pressure: at engine-room floor; from floor to the level of water in the pump well; (b) allowance for friction: force main; suction pipe; (c) total working load upon plungers; (d) steam pressure at the throttle; (e) available clear height above the engine-room floor; (f) vertical distance from engine-room floor to basement floor; (g) wall

to wall of engine room, inside, both directions; (*h*) available space on floor across the engine room; (*i*) available on floor lengthwise of engine room; (*j*) available in basement, lengthwise of engine room; (*k*) distance from building wall to pump well (furnish bidders plans and sections of the building, especially if space is cramped or obstructed); (*l*) air chamber required at the inboard end of the suction pipe; (*m*) air chamber required for the force main; (*n*) capacity; (*o*) number of units; (*p*) transportation and erection facilities, sizes of entrances; (*q*) special local conditions if any.[3]

On cost of pumping engines[3] no really reliable figures can be given. Such figures vary with type, size, head pumped against, steam pressure, supply and demand, and cost of materials. If in doubt as to type needed, get bids on types to be considered. To fix upon cheapest unit for a particular case, consider costs of the following: (*a*) engine, (*b*) foundations, (*c*) land, (*d*) buildings, (*e*) boilers necessary to supply steam to engine, (*f*) coal per ton, (*g*) maintenance and repairs, (*h*) lubricants and waste, and (*i*) any other elements affected by type of engine.

Lubricants. As a cylinder lubricant, flake graphite is used alone or with oils. It fills pores and irregularities of cast iron, and imparts to the surfaces of piston, cylinder, and valves a smooth, dense coating and brilliant polish without apparent pore or crack. Where cylinders "groan" from insufficient, lubrication, application of a little graphite and oil through hand pumps will cure the trouble almost instantly. Where cylinders and valves are scored and cut, flake graphite rapidly fills in and overlays irregularities, restoring surfaces to smoothness. When bearings are grease lubricated, 4 or 5 per cent. flake graphite may be mixed with the grease and fed regularly; 4 to 6 per cent. by weight of graphite well mixed with good machine oil makes a satisfactory proportion, thin enough never to clog the oil ways. One great barrier to adoption of superheated steam has been the difficulty of providing adequate and reliable cylinder lubrication; at these high temperatures flake graphite has proved a reliable remedy for insufficient lubrication. Graphite introduced into a steam cylinder finds its way to the stuffing boxes and packing, and prevents scoring, fluting, or rusting. Any separator, grease extractor, or settling tank that will remove even a fair percentage of cylinder oil will remove all graphite that goes out with the exhaust. Graphite passing into a separator adheres to the baffle plates; remove and clean them from time to time. For certain types of separators it is convenient to purchase duplicate baffle plates which can be changed in a few minutes and the coated plates cleaned at leisure. Specific gravity of graphite is greater than of oil; therefore it is but a matter of time before graphite mixed with oil settles, no matter how finely pulverized or how heavy the oil. Do not attempt to feed graphite mixed with oil in gravity oil cups or sight-feed lubricators; graphite might settle and clog the feed. Have a special oil can for graphite and oil (a heaping teaspoonful of graphite to a pint of machine or engine oil) and shake thoroughly before applying. Perfectly dry graphite will feed through a common squirt can; follow up each application with a few drops of oil to carry the graphite into the bearings. In "splash" lubricated engines, graphite may be added directly to the oil in the crank case, a teaspoonful to a quart.

Water-pump plungers should be lubricated with a waterproof graphite grease. It cannot be washed off, is unaffected by fresh, salt, alkaline, or acid waters, and therefore gives the best service. The graphite in this composition reduces friction to a minimum; it also protects the packing and prolongs its life. For gears, slides and other moving parts, it gives the best results, as it is a tenacious, lasting, and excellent lubricant.

Cylinder Oil, St. Louis Specifications.[15] Cylinder oil shall be a compounded oil of 2 per cent. pure acidless oil, and 98 per cent. pure filtered mineral oil; free from dirt, grit, lumps, and specks; transparent amber in thin film; bright ruby through neck of 4-oz. bottle; translucent greenish by reflected light. It must satisfactorily pass the following tests: Sp. gr., 26 to 28° Bé. at 60°F.; flash point, not below 540°F.; burning point, not below 600°F.; viscosity, not less than 3.75 (Engler) at 212°F.; cold test: must flow readily at 50°F.; water, must not froth or bump when heated in flash cup; tarry and suspended matter: 5 c.c. of this oil shaken with 95 c.c. of 88° petroleum ether in a glass-stoppered graduate must show no precipitation of tarry and suspended matter; volatility: heated for 2 hr. at a temperature of 400°F., this oil must not show a loss of more than 5 per cent. by weight; saponification: when oil is treated with alcoholic potash, it must show a presence of 2 per cent. tallow oil.

Pump Cylinder Calculations. (For table of cylinder capacities, see p. 820.) *Piston Speed.* The ordinary speed for single-stroke pumps is 100 ft. of piston travel per min.; for double-stroke pumps, 140 ft. For feeding boilers, speed should not exceed 50 ft., or 30 to 40 strokes per min. when the boiler is evaporating at its normal rating, 30 lb. of water per hp. per hr. In fire pumps, where the largest quantity is required, the speed may exceed 200 ft. per min.

Table 143. Strokes Required to Reach Piston Speed of 100 Ft. per Min.

Length of stroke, in.	Number of strokes	Length of stroke, in.	Number of strokes	Length of stroke, in.	Number of strokes
4	300	12	100	24	50
5	240	14	86	26	46
6	200	16	75	28	43
7	172	18	67	30	40
8	150	20	60	36	33
10	120	22	55	40	30

Theoretical Discharge. To find the number of gallons delivered per min. by a single-acting pump at 100-ft. piston speed per min. square the diam. of the plungers, then multiply by 4 (roughly).

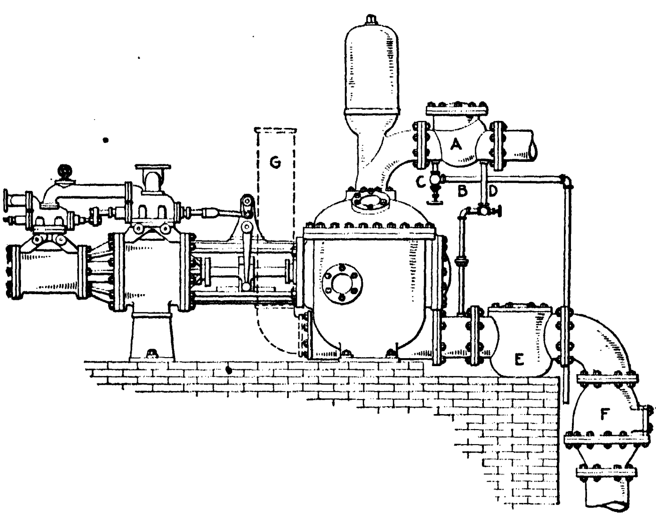
Proportion between steam and pump cylinder is determined by multiplying cross-sectional area of pump cylinder in sq. in. by the resistance on the piston in lb. per sq. in., and dividing the product by the available pressure of steam in lb. per sq. in. The quotient is the cross-sectional area of the steam cylinder. To this must be added an extra area to overcome the friction, usually taken at 25 per cent.

Rule for steam pressure required, when the lift, area* of the water cylinder, and area* of the steam cylinder are known: Take half the lift in ft., multiply

* Area of cross-section.

by the area of the water cylinder in sq. in., add 25 per cent. of this to itself, and divide by the area of the steam cylinder in sq. in. (approximate).

Slip of pumping engines is the percentage difference between theoretical discharge and actual volume of water delivered. Slip is important and easily detected. A Venturi meter on the discharge and a counter on the engine are most convenient and all that are necessary for determination; comparison of volumes pumped, as indicated by each, gives the information.[16] Cause has not been fully understood and extent has been underestimated. Old types of reciprocating pumping engines had short water pistons in long cylinders, and there was little or no provision to compensate for wear due to sand and grit; piston speeds were low and a slip of 50 per cent. around water pistons was not unusual, which could be proved by closing the discharge gate valve while the pump was running, when the speed of the pump would not be reduced to less than one-half normal speed. In later types rotative and plunger speeds are much higher and long plungers are used instead of short water



A—Check valve.
B—Waste pipe.
C—Air valve.
D—By-pass pipe.
E—Strainer box.
F—Suction check valv
G—Vacuum chambe

FIG. 204.—Arrangement of pipe connections, small reciprocating pump.

pistons, frequently "outside-packed." "Slip" past the plunger is generally negligible, but "slip" at the rubber pump valves is often large, due to tardy closing, excessive wear, sticking, and clogging. Many believe that excessive "slip" is always accompanied by noise in the water end, but noise is sometimes absent. If the trouble is due to a relatively *small* number of *bad* valves, there will be noise, but if it results from a *large* number of only *moderately worn* valves, noise can seldom be heard. Sometimes engineers test for "slip" by opening the hand holes underneath the valve decks and allow full water pressure to be exerted upon the tops of the valves. When they find only a small leakage, they erroneously conclude that "slip" at this point is negligible. In centrifugal pumps "slip" occurs in other ways. With crank and flywheel pumps short stroking is negligible, but with direct-acting pumps short stroking frequently exceeds 5 per cent.[17]

In statistics of water unaccounted for, an important cause of inaccuracy is failure to allow sufficient "slip" of pumps. Probably few pumps operate with less than 4 to 5 per cent. "slip" and it is not unusual to find 10 to 30 per cent. Major Gillette and J. C. McLennon in report on water waste in Philadelphia, 1906, place average "slip" at 25 per cent. and maximum at 56 per cent., reducing apparent consumption from 233 gal. per capita to 164. Philips found "slip" in Chicago 10 to 28 per cent., which he was able to reduce, by packing and repairing, to about 5 per cent. Evansville, Ind., 1903, cut its apparent consumption from 200 to 107 gal. per capita by repairing pumps.[18]

ROTARY PUMPS*

Principle. There are two pistons, with intermeshing teeth, or swells, revolving in a pump case so as to occupy successively the whole space, and in this way to displace the water. By the revolution of these pistons the water is continuously sucked in and discharged. No suction or discharge valves are required. This type of pump is used in most mobile fire apparatus and in a few stationary installations, notably Connorsville, Ind.

Advantages.[4] Small size compared to reciprocating pumps of same capacity; fair flexibility; high efficiency over a wide range of life, although this decreases with wear and slip; no need of priming; quick starting not affected by long periods of idleness; uniform flow; positive action; adaptability to any lift or capacity by varying speed; first cost lower than triplex pumps.[6] A stock pump can be used for any lift or capacity, by altering the speed. Efficiency of a new pump is high over a wide range of lift.

Disadvantages. Position close to water; high first cost; poor durability; low speed, so that electric motors cannot be direct connected; limited head; close adjustment, to prevent excessive slip; poor performance with long shafts;[19] large size compared to centrifugal pumps. Efficiency decreases with wear and slip.

CENTRIFUGAL PUMPS†

Principle. Water enters near axis of the rapidly rotating impeller and flows outward to the periphery, which it leaves at a high velocity. The height of water column which would produce an equal velocity is the theoretical total head which this type of pump will develop. Centrifugal force plays an important part in changing the mechanical work supplied to the shaft to kinetic energy. Correct speed is most essential to proper operation of a centrifugal pump, and prime movers should be selected on this basis.

Adaptability. Centrifugal pumps are ill adapted to drive by steam engines, because of high speeds required. Modern electric motors and steam turbines have made centrifugal pumps preeminent in waterworks practice. Water turbines,[20] gas engines, and Diesel engines have also been used to drive

* Makers of rotary pumps include: Blackmer Rotary Pump Co., New York; Kinney Mfg. Co., Boston; Northern Pump Co., Minneapolis; Nash Engineering Co., South Norwalk, Conn.; Rumsey Pump Co., Seneca Falls, N. Y.

† Great varieties of pumping units are offered by a large number of makers; many of them are quite satisfactory for usual requirements. Much additional useful information may be had from the publications of the best manufacturers. Reference is also made to "Centrifugal Pumping Machinery," C. G. De Laval (McGraw-Hill Book Company, Inc., 1912); "Centrifugal Pumps," Loewenstein and Crissey (Van Nostrand, 1911); "Centrifugal Pumps," R. L. Daugherty (McGraw-Hill Book Company, Inc., 1915); "Centrifugal Pumps," J. W. Cameron (Scott, Greenwood, London, 1921); "Modern Pumping and Hydraulic Machinery," E. Butler (Griffin, London, 1922), and also the various handbooks.

centrifugal pumps. Improvements[21] have been made in recent years in the adaptation of impellers to conditions of head and discharge, in methods of balance, in lubrication, in the reduction of clearance, and in accessibility to the impeller. Under heads in excess of that assumed in design, efficiency and discharge of a pump will be lower; "at shut-off" head, there will be no discharge. At heads lower than designed for, greater discharge and less efficiency, but not necessarily less power consumption, result.

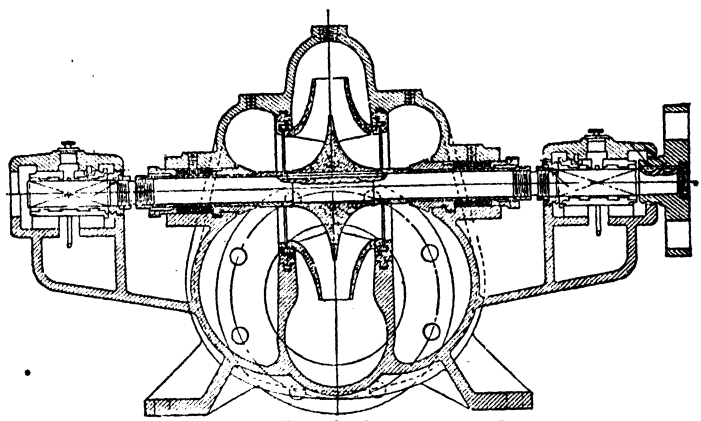


FIG. 205.—Typical section of DeLaval single-stage, double-suction centrifugal pumps of small and moderate size.

Volute-type (termed "non-diffuser" and also "centrifugal") pumps are used for heads below 100 or 150 ft. The water leaves the impeller in a direction causing friction losses which become important under high heads. The casing is in the form of a spiral, or volute, in order that the pump may discharge around the entire circumference and also that some of the velocity head may be converted to pressure head. This type costs less than the turbine, and has a greater use. It is less efficient than the turbine type and is recommended only for large volumes at low heads.[22]

Turbine-type centrifugals are used for higher heads and are characterized by constant cross-section of casing, and by diffusion vanes† external to the impellers which discharge the water tangentially to the casing so that friction is minimized. The slender vane edges, essential to high efficiency, readily bend, batter, and erode, thus lessening their effectiveness. Diffusion vanes should be smooth and easily replaceable.* These pumps suffer somewhat in efficiency when operated below rated head and speed, as changed conditions increase the obstruction that the diffusion vanes offer to water; some makers[23] of both types state that changes in operating conditions affect one type as much as the other. Turbine pump requires fewer stages than the volute type to deliver water against high heads. The small waterways promote choking. Turbine pumps are heavier than volute, and cost more, but are more efficient if properly proportioned.

Specifications covering design, equipment, installation, and operation of centrifugal pumps for fire service have been issued by National Board of Fire

* See York, *J. A. W. W. A.*, Vol. 5, 1918, p. 118.
† Recent designs favor double suction in preference to diffusion vanes.

Underwriters (1915) and Associated Factory Mutual Fire Insurance Companies (1923).

Advantages.[4] Low first cost and operation cost; excellent durability; small weight; compactness; low cost for foundations and house; simplicity in design and operation; quick-starting; steady flow; high speed for direct connection to high-speed drives; starting torque rarely exceeds 30 per cent. of normal full-load torque; but one moving part; freedom from shock; pressure not excessive in case of stoppage;* low rate of depreciation; operation in parallel for quantity, or in series for pressure boosting.

Disadvantages. To obtain high pressures, multi-staging is necessary; less flexibility; necessity for priming; efficiency is lower than that of a good reciprocating pump; direct connection to low-speed engines not possible for high-pressure pumping; many operators are more familiar with reciprocating pumps; above 150 hp., centrifugals cannot equal in duty the modern triple-expansion pumping engines,[24] coupled with the latest-type steam engines, but are cheaper to run, particularly for intermittent service; rate of discharge cannot be efficiently regulated for wide ranges in duty; will not operate with slight leakage into the suction; other types of pumps handle higher suction lifts more efficiently; will not operate efficiently where there is a constant variation in pumping head;[23] the high speed requires accurate balance and frequent attention to bearings; in case of break in line, pump may race and damage the motor; they lack the convenience of cylinder pumps for gaging the quantity pumped.

Series and Parallel Operation. Two single-stage pumps, each of a capacity of 500 g.p.m. against 75-ft. head will, if connected in series, deliver 500 g.p.m. against 150-ft. head, or, if connected in parallel, deliver 1000 g.p.m. against 75-ft. head. If two pumps discharge into the same main, their characteristics should be the same or one pump may be cut out altogether. In plant at Edmonton,[84] twin-series pumps operate singly at 900 r.p.m. to deliver 7000 g.p.m. against 110 lb. pressure, and in case of fire are put in series to deliver 7000 g.p.m. against 160 lb.

Horizontal vs. Vertical. Horizontal pumps have higher efficiencies, can be more easily direct-connected with motors or prime movers, and cost less. Difficulty of maintaining the alinement of vertical pumps makes them inadaptable to high speeds. Vertical-shaft pumps operate on heads of 30 to 50 ft. per stage.[21] The practice of using a single stage for heads as high as 90 ft. was discarded as impracticable, on account of the effect of the excessive speeds required on the vertical shafts. Where there is a large fluctuation in water level, there may be troubles in priming a horizontal pump. To minimize priming troubles, pumps are often placed in "dry wells" below the level of low water in the suction well. "Dry wells" are commonly damp; motors will deteriorate under such conditions. If vertical pumps are adapted for such conditions, economy results both from the smaller size of the "dry well" and the longer life of the motors. Vertical pump requires only suspension thrust bearing; if placed above the vertical motor, it is readily accessible.

Stages. The velocity imparted to the water determines the height to which it can be raised. Naturally, there are mechanical limits to the r.p.m.,

* Throttling discharge merely decreases flow without materially increasing load.

which determine pumping heights. To overcome this, several pumps are installed on one shaft, the suction level of the second pump being at the pressure head against which the first pump is working. The second pump impresses an additional pressure head, and so on through as many stages as required. Single-stage pumps are made with single or double suction, multistage with single suction only. A single-stage pump should not be used much above 100 ft., although one delivered 21,000 g.p.m. against 412-ft. total head at Lynchburg, Va.,[25] and was cheaper in first cost than a three-stage volute or a two-stage turbine pump. For fire service, National Board of Fire Underwriters specifies not less than two nor more than four stages.

Casings. Pumps above 3 in. have cases split along center line, with all pipe connections and shaft bearings in lower half, thereby permitting access to running parts on removing top half of casing. Turbine pumps not split along

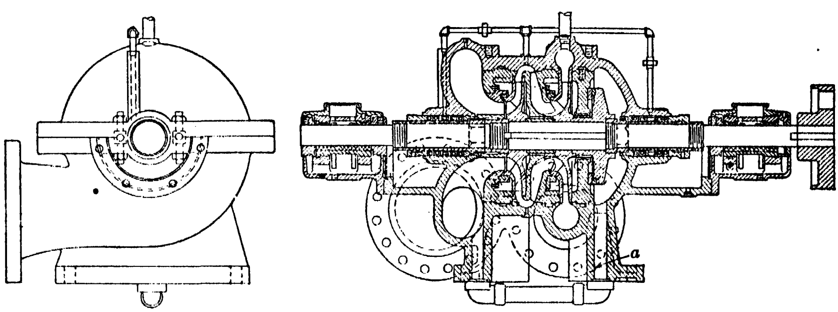


FIG. 206.—Worthington 8-inch, 2-stage turbine pump, Underwriter Fire Service. *a*, non-throttling orifice.

center are likely to rust, so preventing removal of diffusion rings and guide passages.

Impellers should be bronze to avoid corrosion; gun metal (88 per cent. copper, 10 per cent. tin, 2 per cent. zinc) is recommended. In order to reduce skin friction and surface eddying, impellers should be machined both inside and out. Inaccessible interior surfaces should be scraped and draw-filed to as high a polish as practicable. Impellers should be statically balanced by testing with knife edges, and their centrifugal balance made sure by idle rotation at rated speed. Sides of impellers on water pumps* should always be closed to avoid friction of water with pump casing, and to afford an actuating force to all water by eliminating clearance at side of vanes. Vanes should always be fitted to templets to insure accuracy of angle and balance. In Roturbo pumps, of which Rees[26] and Manistee are typical, the impeller has the form of a large-capacity drum, to form a pressure chamber to generate pressure head directly.

Impeller Packing Rings. Figure 207 shows the two types in common use. Economical results are obtainable from both. Packing rings should be hard bronze. One prominent manufacturer uses a composition (5 parts tin, 1 part copper) which is said to have been selected after careful experiment.

Shafts. Impeller shafts should be steel, if quality of water to be pumped permits, or bronze for salty, acidulous, or alkaline water. Steel shafts should

* Sewage pumps require open impellers.

be bronze-jacketed always, for protection against corrosion and to permit repairs; they should be of high-strength steel to stand the stress, and especially to afford rigidity in pumps of high rotation. Shafts and shaft jackets should be provided with straight keys to rotate impellers; should be so made that a nut holds impeller against a shoulder to prevent impeller from drifting along the shaft; should have collars near ends of bearings to clear off oil by centrifugal force and prevent it from creeping out of bearing and wasting.

FIG. 207.—Two types of impeller packing rings.

Bearings. Bearing sleeves should be easily removable without disturbing the other parts of the pump. All oil bearings should have ring oilers and deep wells to permit sedimentation. Interior bearings of hard bronze are placed between impellers and at bottoms of stuffing boxes; these are water lubricated only. Small pumps and large low-speed pumps are frequently made with lignum vitæ bearings with water lubrication; they should be avoided, as they have proved unsatisfactory in service. The quality of oil required is the same as for steam turbine.

Thrust Bearings. A centrifugal pump is never wholly balanced against end thrust. Pumps with double suction and single outlet are nearest to this realization, but collars must be provided on shaft to keep impeller in proper position inside casing and to take thrust resulting from unequal flow of the two inlets and inequality of areas of passages. Liability to end thrust is greatest in the multi-stage classes. The latest development in theoretically balanced multi-stage pumps is the design of impellers with baffle rings front and back, with holes in the back of each impeller so as to equalize the pressures. This remedy is only partially successful, as it can be used only in conjunction with a thrust bearing. To hold the pump parts in correct relation, thrust bearings of the marine, ball, roller, and piston type are all in successful use. In all types at least one orifice is between a moving member and some fixed portion of the pump. Such orifices should be bronze-lined for bearing purposes and to prevent scoring resulting from high velocity.

Bed Plates and Foundations.* Foundations should be of sufficient depth to support the pump base rigidly, and should be located in accordance with the foundation plans of the pump, but sufficient allowance should be made for lateral adjustment of the bolts. Bed plates should be heavy, so designed as to support both pump and motor, and fitted with dowels for their accurate alinement. Bed plates should be deep and provided with openings in the top through which concrete and grout can be admitted after alinement. By filling the interior, noise and vibration are materially diminished. When steam turbines or motors and pumps for large units are purchased from differ-

* See "Machinery Foundations and Erection," by Croft (McGraw-Hill Book Company, Inc., 1923).

ent manufacturers, the bed plates are frequently separate; if so, the foundation should be large. After the bed plates have been bolted in position, and after the pump and motor have been placed in their final positions, the unit should be carefully tested for alinement by inserting a machinist's thickness gage between the coupling flanges and by making sure that the whole shaft revolves freely when coupled together. The bed plates should then be rigidly grouted in place and the pump and motor doweled to the bed plates.

Fittings for Centrifugal Pumps. Since high velocity through the pump chambers is essential to efficiency of centrifugal pumps, the chambers are relatively small. To keep friction losses in the piping within reason, piping is of larger diameter, with increasers connecting to pump inlet and outlet.* These increasers are generally short, of special design to save space. Often room is gained by using an increasing quarter bend. Eliminate all bends possible in piping; where pumps are in a pit, piping should be carried out at 45° rather than vertically, as thereby one 90° bend is eliminated. Velocities in piping should be kept below 6 or 8 ft. per sec.[81]

Important Data for Pump Designers. Each impeller is special and great care must be exercised to furnish the pump builder with all important data:

1. Number of pumps required and nature of service.
2. Is vertical or horizontal type of pump desired?
3. Capacity required in g.p.m. (U. S. or imperial).
4. Maximum suction lift and distance from source of supply.
5. Length and size of suction and discharge pipes, giving number and kind of elbows.
6. Total pumping head—suction and discharge head, plus pipe friction under worst conditions.
7. If pumping head is variable, what is maximum variation for both suction and discharge?
8. If pump is located below source of supply, will it be submerged or placed in a dry pit?
9. Will pump be required to operate continuously? If not, at what intervals?
10. Nature of liquid to be pumped—hot, clear, fresh, alkaline, cold, gritty, salt, or acidulous; give temperature and specific gravity.
11. If pump is to be driven with electric motor, give current characteristics; if direct current, give voltage; if alternating current, give phases, cycles, and voltage; give type of electric control desired.
12. If pump is to be steam driven, is reciprocating engine or steam turbine desired? Give steam pressure, superheat, if any, condensing or non-condensing; if condensing, give vacuum.
13. If pump is belt driven, give speed and diameter of driving pulley, if possible.
14. Give direction of rotation.

Prime Movers.† Rotating speeds up to 1500 r.p.m., continuity of water flow, absence of shock and noise, and economy of space make centrifugal pumps particularly adaptable to motor drives.[27] For driving by internal-combustion

* To deliver 400 g.p.m. through 300 ft. of 5-in. pipe will add to pumping charges $375 per year, over those with a 10-in. pipe. The larger pipe would pay for itself in a year.[81]

† See also p. 520.

engine some form of speed-increasing gear is required. Where centrifugal pumps are driven by reciprocating engines, higher efficiencies can be obtained and cost of installation reduced by using high-speed pumps driven through speeding-up gears of the double-helical type.[28]

Operation.* Since a centrifugal pump depends upon speed to deliver rated capacity against rated head, this speed should be maintained. Great care should be given the shaft stuffing boxes, especially those on the suction end, to see that no air enters. When water-seal lanterns† are used in stuffing boxes, the gland bolts should be drawn up tight at first and then released so that they will be fingertight and allow a small leak of water through the gland. This method secures long life for packing and reduces liability of cutting the shaft. So long as one can see water emerging from the stuffing box, it is safe to assume that no air is entering.[4] When throttling is necessary, use valve on suction side, as the power consumed is less than when the discharge valve is throttled, although the efficiency is decreased. Before starting new or unused centrifugal pumps, it is advisable to clean out the bearings thoroughly, including the thrust bearing, by pouring in kerosene and allowing it to run out at the bottom, as dirt is liable to get in during shipment or idleness. The bearings should then be filled as full as possible with first-class lubricating oil similar to turbine oil. In order to discover whether a pump is operating under the conditions for which it was sold, very little investigation is required. The dynamic head is readily obtained by a vacuum gage on the suction side and a pressure gage on the discharge; both at the pump openings. The sum of simultaneous readings of these gages plus the vertical distance between their centers gives the dynamic head. Should the water flow to the pump under pressure, this pressure must be deducted from the discharge pressure and then the difference between the centers of the two gages should be added.

Troubles. If, on starting a centrifugal pump, after it has been thoroughly primed, it is found that only a small quantity is discharged or none at all, air is probably present due to defective suction-pipe joints or excessive suction lift. Tight suction lines are essential. Tests at University of Pennsylvania[29] showed that small percentage of air had little effect on efficiency, but a noticeable effect on capacity. Should the pump suddenly fail to discharge its full capacity, this trouble is due to the fact that the water in the suction well has receded lower than the suction lift, or the end of the suction pipe has become exposed. If the suction pipe is relatively small, thus having high velocity in it, its end need not be entirely uncovered, because air can be drawn down through the water by the whirlpools formed, and for this reason the bottom of the suction pipe should be at all times at least four diam. below the surface of the water. A leaky suction pipe must be immediately investigated and made tight. If a centrifugal pump discharges a fair quantity at its nozzle but fails to deliver the rated quantity under rated head, it is probable that the rotative speed is too low; the name plate should be consulted to make sure that the requirements called for are being fulfilled. Pumps should be opened from time to time to make sure that no foreign matter is lodged in the impeller or passages and no undue corrosion is taking place.

* See also "Check Valves," p. 451.
† See p. 499, Stuffing Boxes.

Priming. A centrifugal pump will not operate until its pump case is full of water, *i.e.*, all air is exhausted. Priming is accomplished in various ways. Where practicable, it is well to put the pump below the lowest level of the influent well, so that the casing fills by gravity. Even for such an installation, mechanical priming is desirable, as a small air leak may interfere with pump operation. Priming may also be accomplished by some form of steam ejector, or by many patented devices.* Simplest way is to close the valve in the discharge and fill the system, when there is a foot valve on the suction pipe. In many cases, foot valves are not practicable nor desirable, and it is then necessary to close the discharge valve and prime with a vacuum pump or ejector. When foot valves are used, it is necessary to see that they are of the free-opening, clapper type with area through the valve sufficiently great to avoid unnecessary resistance. After vacuum priming, pump may be brought to rated speed with discharge valve closed, and then pump put into service by opening discharge valve. Air or other gases in suction pipe or well will reduce delivery and efficiency. Care should be taken to eliminate air before it enters suction pipe by using a system of baffling.

Noise in Pumps. Noise in a centrifugal pump is an indication of impact; impact means poor design. Specifications should forbid noise, and the purchaser should require the builder to eliminate it, as it is an indication of wear on impeller and on casing. Noise of impact is a rattling and pounding, but should not be confused with the hum due to rotation at high speed. Objectionable noise may be caused by air in suction line or leakage through stuffing boxes.

Dimensions. Floor space occupied varies widely with different makes. To indicate limits, not to endorse specific makes, Tables 144 and 145 are offered. Table 144 applies to Alberger one-stage pumps driven directly by motors; Table 145 to Cameron multi-stage pumps with turbine drive. Cameron Class "MT" and "ST" pumps. "D" and "I" will vary with the stage. The additions listed in Table 146 include the variation per stage and should be multiplied by 2 for four-stage pumps and by 3 for five-stage pumps.

Relation of Capacity, Speed, Power, and Head. Theoretically, capacity produced by an impeller varies directly as peripheral speed; head produced, as square of speed; and horsepower required, as cube of speed.

$$\frac{Q_a}{Q_b} = \frac{N_a}{N_b} = \frac{H^{\frac{1}{2}}_a}{H^{\frac{1}{2}}_b} = \frac{P^{\frac{1}{3}}_a}{P^{\frac{1}{3}}_b}$$

where Q is discharge at N r.p.m., against H-ft. head, with required power P. Assume a pump (running most economically) is discharging 7000 g.p.m. (Q_a) against 60-ft. head (H_a), at 800 r.p.m. (N_a) and consuming 140 hp. (P_a). How are the characteristics altered by increasing speed (N_b) to 950 r.p.m.?

$$Q_b \text{ becomes } 7000 \times \frac{950}{800} = 8312 \text{ g.p.m.}$$

$$H_b \text{ becomes } 60 \times \frac{950^2}{800^2} = 84.6 \text{ ft.}$$

$$P_b \text{ becomes } 140 \times \frac{950^3}{800^3} = 234.3 \text{ hp.}$$

* Among these are Apco, made by Automatic Primer Co., Chicago.

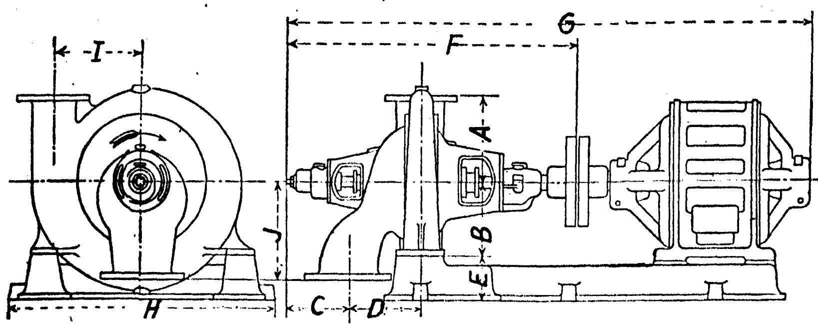


FIG. 208.—Dimensions of Alberger one-stage pumps (Table 144).

Table 144. Approximate Dimensions, Single-stage Volute Pumps, Motor-driven (Alberger)

	Size, inches		Maximum capacity G.p.m.	Dimensions in inches									
	Suction	Discharge		A	B	C	D	E	F	G	H	I	J
Dimensions E and G, are approximate, and will vary with size and make of motor. Do not use these dimensions for construction work.	2	1½	65	6½	5½	7¼	5¼	6	25¼	46½	20¼	6¾	6½
	2½	2	100	7¼	6	6⅞	5¾	6¼	27⅛	48½	20¼	7½	7
	3	2½	150	9	8	7¼	6½	6½	30⅞	54	24½	10	8½
	4	3	250	8¾	7½	7¼	7	6¾	30	58	25¾	9¼	9
	5	4	450	10	8½	7¼	8	7½	34¼	64½	30¾	11	10½
	6	5	700	11½	9¾	7¾	9½	8	38¼	71¾	35½	12¾	12½
	8	6	1000	13	11	8½	10¾	8½	44⅜	79¾	38½	13⅞	14½
	10	8	1800	14½	14	9	14	9	46½	86½	44¼	16⅜	16½
	12	10	2800	17½	15½	9½	15¾	10½	55½	104	55½	19	18
	14	12	4000	20	17½	10½	18½	11	60	115	65½	22½	21
	16	14	5500	22	20	11¾	20½	12	67¾	123	71¾	25⅜	24
	18	16	7500	23	24	11¾	22¼	12	72½	132	78¼	28¼	26½
	20	18	9500	24	28	13¾	25½	12	80¼	146	84¼	31¾	29½
	22	20	12000	29	29½	14¾	27	14	88	171	95¼	35¼	31½
	26	24	18000	32	30	16¾	31⅞	14	104	180	120	42	36

Specific speed $= \dfrac{N \times \sqrt{\text{g.p.m.}}}{H^{\frac{3}{4}}}$. Pumps of low specific speed are inherently of low efficiency (see Sherzer's development in *E. N. R.*, Oct. 4, 1923, p. 561).

Impeller diam. is fixed by peripheral velocity. A pump running too fast for the head generated cannot be efficient. There is a definite relation between shut-off pressure and working head that gives best efficiency; this should be held within close limits. When pump is operated at higher speed than is consistent with conditions, cavitation may be caused, resulting in erosion of

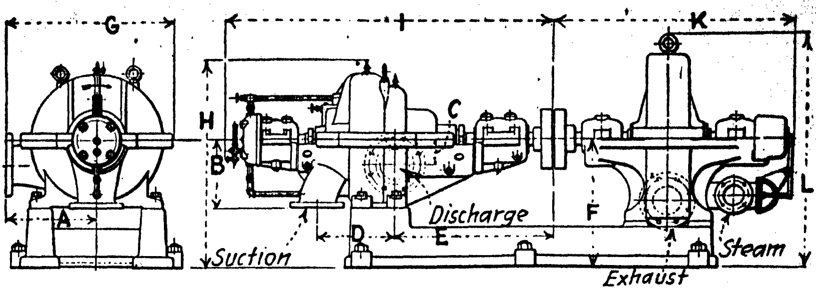


FIG. 209.—Dimensions of Cameron multi-stage pumps (Table 145).

Table 145. Approximate Dimensions, Multi-stage Centrifugal Pumps with Direct-connected Driver (Cameron)

Number	Suct. dia.	Disch. dia.	A	B	C	D	E	F
2½	3″	2½″	12½″	9″	2″	9¼″	23¼″	16″
3	4″	3″	14″	10½″	2½″	11⅜″	2′-2⅞″	19″
4	4″	4″	15¾″	11½″	3″	12⅝″	2′-4⅝″	21″
5	5″	5″	16¾″	12⅞	3⅝″	14¼″	2′-7″	23″
6	6″	6″	18½″	14″	5″	16¼″	2′-8⅞″	2′-2″
8	8″	8″	22″	16¼″	5¼″	19¼″	3′-1⅝″	2′-4″
10	10″	10″	2′-0″	19¾″	9¼″	22¼″	3′-8″	2′-6″
12	12″	12″	2′-6″	2′-1¼″	10″	2′-0¾″	4′-3″	3′-3″
14	14″	14″	2′-10″	2′-5″	12″	2′-7″	5′-0⅜″	4′-1″
16	16″	16″	3′-2½″	2′-10″	14″	2′-9⅝″	5′-10¾″	4′-9″
18	18″	18″	3′-6½″	3′-2½″	16″	3′-3″	6′-8″	5′-6″

Number	Suct. dia.	Disch. dia.	G	H	I	K	L
2½	3″	2½″	22⅞″	2′-1¾″	4′-0 7/16″	2′-9¼″	2′-4″
3	4″	3″	2′-1⅞″	2′-6″	4′-7 3/16″	3′-5½″	2′-10″
4	4″	4″	2′-4¾″	2′-9¼″	4′-9 13/16″	3′-5½″	2′-11″
5	5″	5″	2′-7¼″	3′-0″	5′-4⅛″	4′-0″	3′-2¾″
6	6″	6″	2′-10¼″	3′-6″	5′-7⅞″	4′-0″	3′-4″
8	8″	8″	3′-4½″	3′-10″	6′-5 5/16″	4′-0″	3′-6″
10	10″	10″	3′-9⅞″	4′-0″	7′-6″	5′-6″	4′-6″
12	12″	12″	4′-5″	5′-3″	8′-2¾″	7′-0″	5′-10″
14	14″	14″	5′-5⅛″	6′-8″	10′-8½″	8′-0″	6′-9″
16	16″	16″	6′-3½″	7′-9″	12′-2¼″	10′-0″	7′-8″
18	18″	18″	7′-2″	9′-1″	13′-8″	10′-6″	8′-0″

Table 146. Additions per Stage, Table 145

Size	Add to "D" and "I" per stage	Size	Add to "D" and "I" per stage
2½	4 ″	10	10 ″
3	4⅞″	12	12 ″
4	5⅝″	14	14½″
5	6 ″	16	16 ″
6	7 ″	18	18 ″
8	8½″		

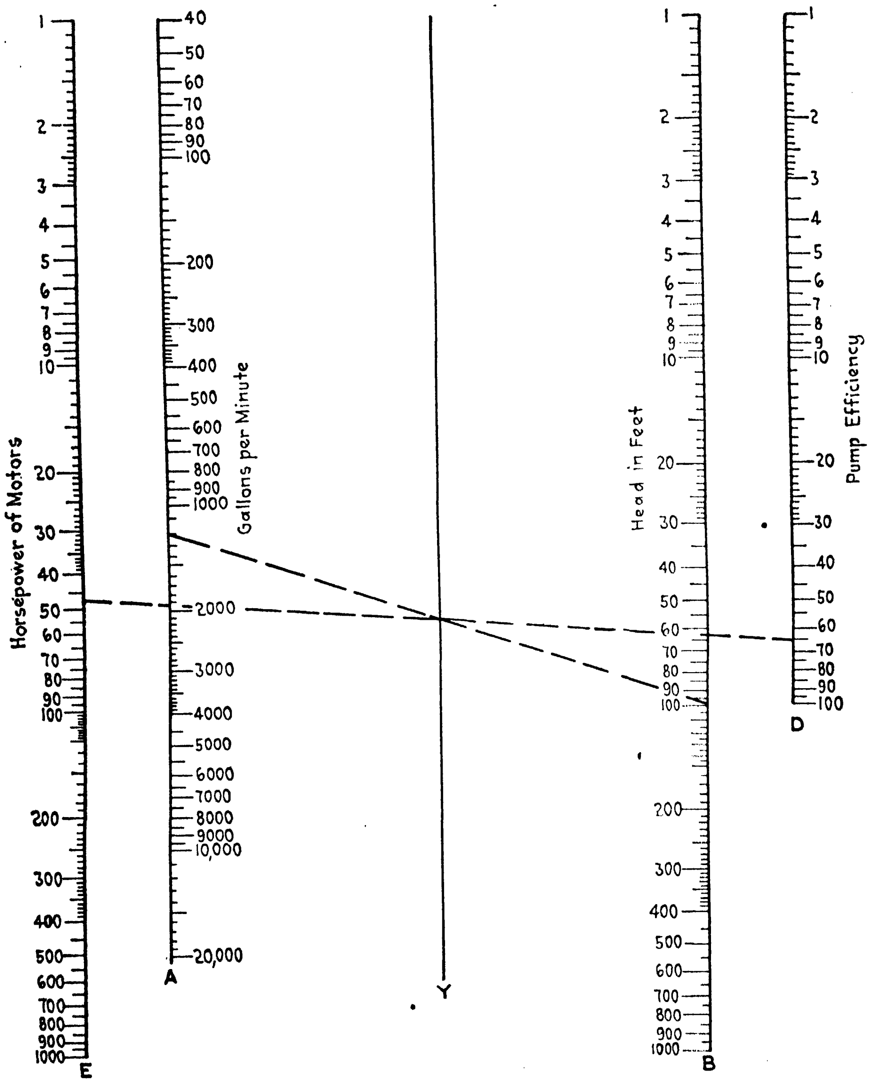


FIG. 210.—Centrifugal pumps. Determination of motor horsepower.
(Earle Gear & Machine Co.)

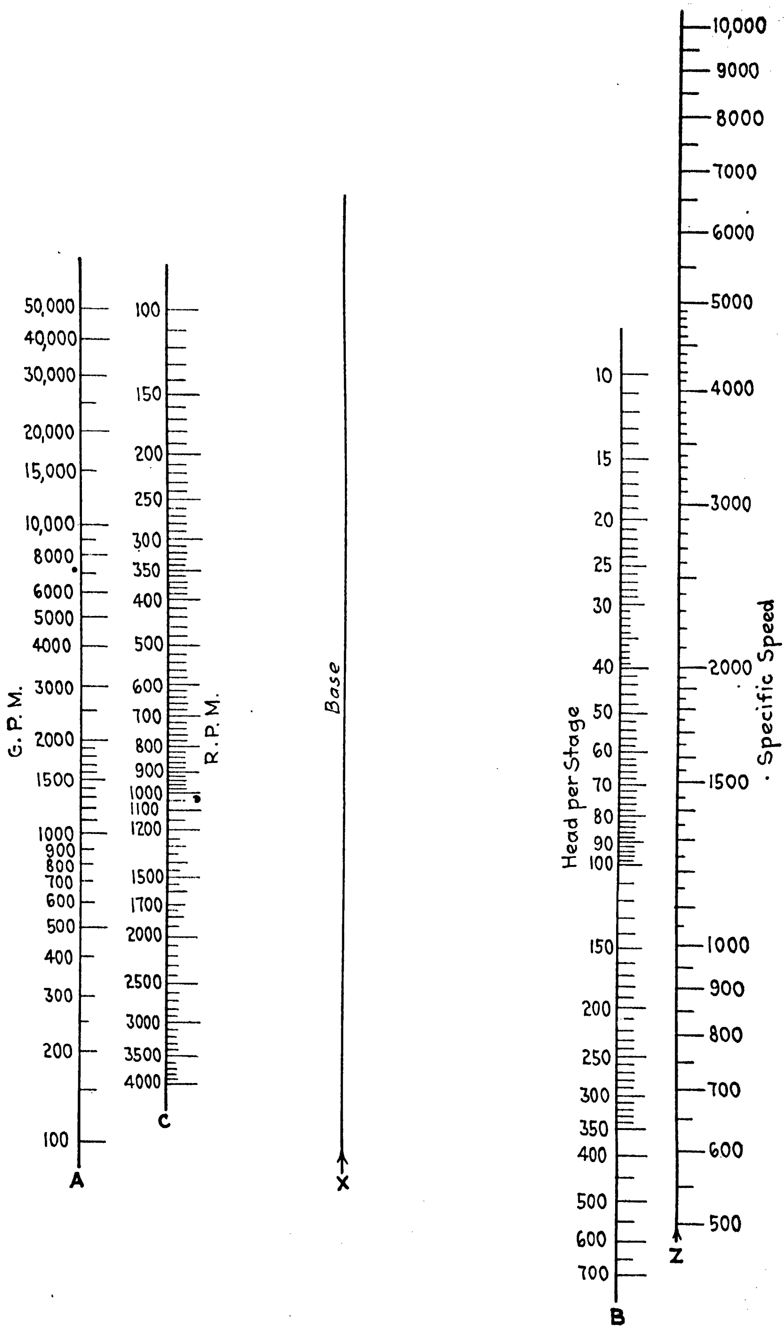


FIG. 211.—Centrifugal pumps. Specific speed chart.
(Earle Gear & Machine Co.)

impeller. Excessive speed also tends to destroy bearings and to cause "fatigue" breaks of the shaft.

Brake Horsepower.[30] Having given the g.p.m. and head in ft., connect scales *A* and *B* (Fig. 210) by a line and mark the point of intersection with reference line *Y*. From this point connect with that point on scale *D* corresponding to the assumed efficiency, and read brake horsepower on scale *E*.

Probable Efficiency. Place ruler across given values on scales *A* and *B* (Fig. 211) and mark point of intersection with base line *X*. From this point place ruler to pass through given value on scale *C*, and read specific speed from scale *Z*. With specific speed and g.p.m. determined, ascertain probable efficiency from Fig. 212.

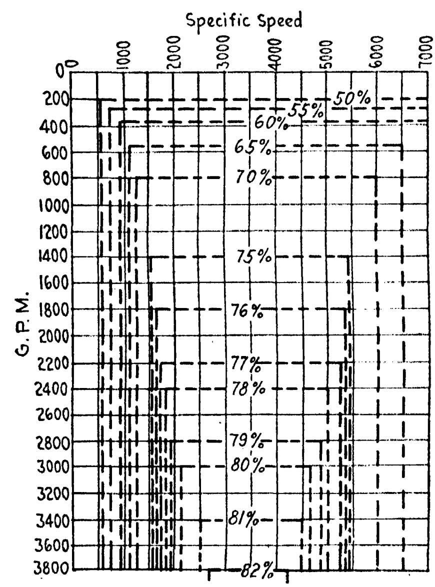


FIG. 212.
(Earle Gear & Machine Co.)

Pump Characteristics. Relation between head, capacity, speed, and horsepower required to drive a centrifugal pump can be expressed by curves plotted either from calculations or from test data; these curves represent the characteristics of the particular impeller chosen.* Without altering the pump casing to any great extent, any one of a variety of impellers each having separate and distinct characteristics may be used, so that for any given size of pump, capacity may remain constant, while head, speed, and horsepower required are varied over a wide range. Typical characteristic curves are shown in Fig. 213. In these curves, capacity, total head pumped against, electric horsepower, and efficiency are all shown when the pump is operating at a constant speed. For instance, the capacity required is 40 m.g.d.; a vertical line from 40 on the bottom scale intersects the curves, at ordinates corresponding to total head pumped against of 224.6 ft.; electric hp. required to operate of 1960, and pump efficiency of 82 per cent. This is not efficiency of

* See Sherzer's new method of determining efficiency, *E. N. R.*, June 30, 1921, p. 1114.

the unit (termed "wire to water" efficiency for electric drive) but means that a motor which will be capable of delivering at least 1960 electric hp. at the shaft will be required, and that 82 per cent. of this electric horsepower will be converted into water horsepower by the pump. With some impeller designs, horsepower required when operating against heads lower than that for which the pump was designed increases so rapidly that serious damage may be done to the driving motor, especially if the motor is of the constant-speed type, while with others it is possible to reduce the operating head to a minimum without overloading the motor beyond the safe limit allowed by motor builders. The latter condition exists in fire pumps, where variation in num-

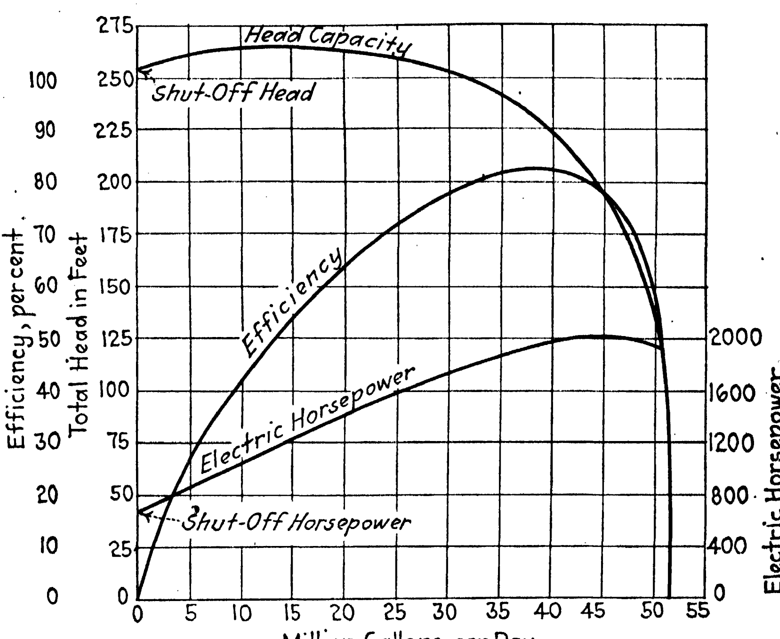


FIG. 213.—Characteristic curves of 30-inch Worthington Volute pump. Constant speed, 615 r.p.m.
(Official test of Montreal pumps, June, 1921.)

ber of streams or bursting of hose would be disastrous, unless the impeller were designed for such emergencies.

Characteristic curves should be studied in arriving at size of motor. In Fig. 213, for instance, the pump requires approximately maximum horsepower at point of maximum efficiency, and a motor selected for normal operating conditions could not be overloaded. Were the high point of the power curve to fall well to the right or left of the high point of the efficiency curve, it would be possible at times to overload the motor, and a size to meet these heavier conditions should be selected.

Efficiency. In Fig. 215 are curves showing relation between maximum efficiency and capacity of volute pumps. As this includes all sizes and quan-

tities, it cannot be taken as an average, but is true when head and speed are in their best relation. Figure 214 goes further into detail and shows the relation of quantity, speed, and efficiency for 6-, 8-, and 10-in. double-suction, single-stage volute pumps. The dividing curve is in each case common to

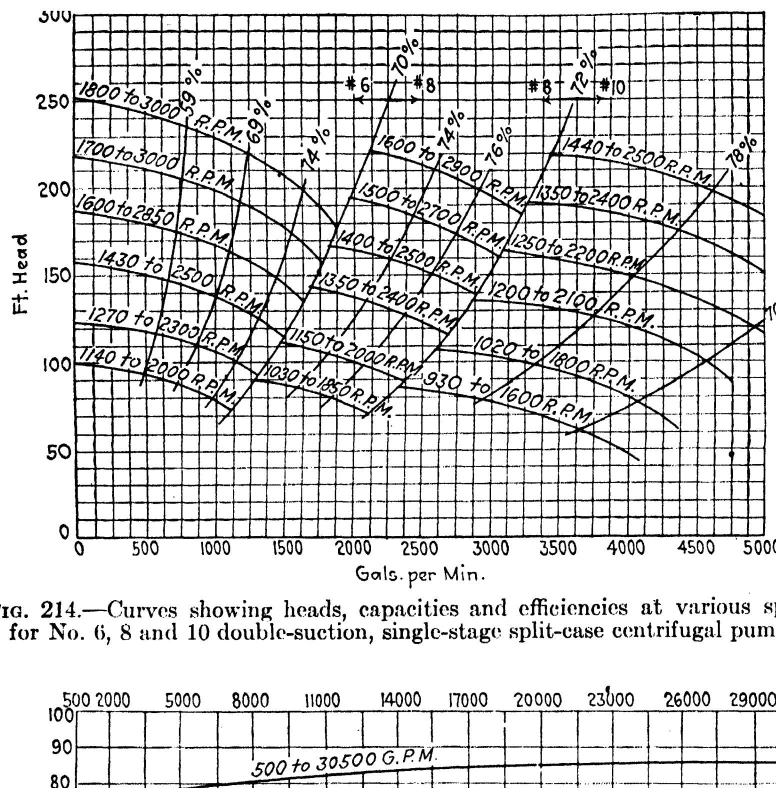


FIG. 214.—Curves showing heads, capacities and efficiencies at various speeds for No. 6, 8 and 10 double-suction, single-stage split-case centrifugal pumps.

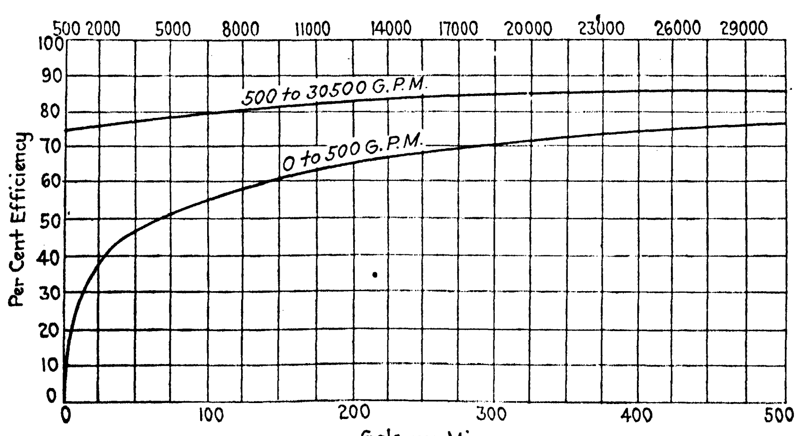


FIG. 215.—Curves showing relation between maximum efficiency and capacity.

pumps on either side. Each manufacturer of centrifugal pumps has a stock line which is usually not modified for speeds other than that for which their impellers are designed; the customer is therefore furnished a pump which would be more efficient if the speed were better related to head.

Efficiency varies with the capacity. A small change in the speed under constant head has a marked effect on the efficiency and less effect on the capacity. Constant speed is desirable (see p. 491). In small sizes, disproportionate water-friction losses reduce the efficiency. Efficiency varies from 30 to 40 per cent. in small pumps up to 75 to 85 per cent. in large. Consult manufacturers' rating tables. For power requirements, reduce these figures by efficiency of driving unit. Motor-driven centrifugals have shown a combined efficiency as high as 82 per cent.[7]

Performance. Centrifugal pumps with electric drive are often purchased on performance specifications. See Minneapolis requirements in *J. A. W. W. A.*, Vol. 7, 1920, p. 88.

STEAM TURBINES*

Use in Waterworks. The steam turbine has been developed into a most economical mechanism for driving large centrifugal pumps; combination is

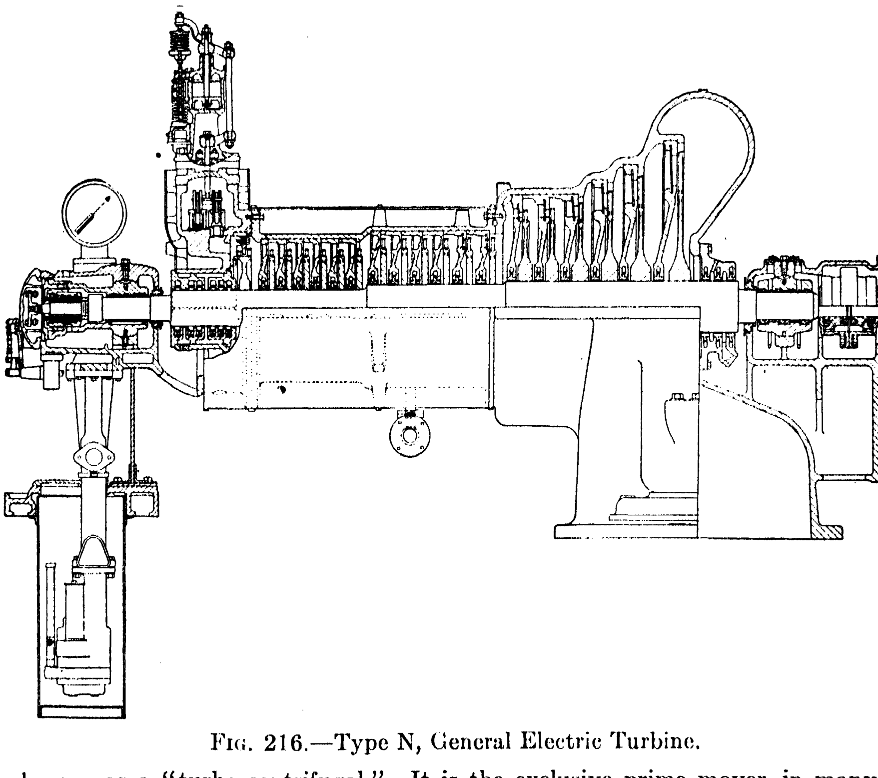


FIG. 216.—Type N, General Electric Turbine.

known as a "turbo-centrifugal." It is the exclusive prime mover in many large pumping stations. It combines maximum efficiency of power transmission with simplicity, compactness, reliability, and small weight. Turbines are admirably adapted to high steam pressures, high superheat, and high vacuum. Boiler-room economies will benefit turbines more markedly than they will reciprocating engines. For pumping small volumes against high

* Steam-turbine manufacturers include De Laval Steam Turbine Co.; Midwest Engine Co.; General Electric Co. (Curtis); Kerr Turbine Co.; Westinghouse Co. (Parsons); Moore Steam Turbine Co.

heads where exhaust steam can be utilized and low steam consumption is not of great importance, centrifugal pumps can be connected directly to, and run at same speed as, turbines. Under other conditions speed-reducing gears must be employed whereby turbine and pump are each operated at most efficient speed. (Centrifugal pumps run at excessive speeds, do not conform to their hydraulic design, and give poor mechanical performance.) A steam turbine can be operated under greatly varying conditions due to change in steam pressure, superheat, and vacuum (see Fig. 218).

Regulations covering steam turbines driving centrifugal fire pumps have been issued by National Board of Fire Underwriters and Associated Factory Mutual Fire Insurance Companies, 1923. Specifications should require a bidder to submit list of successful installations, of design and size comparable to those on which bids are to be received.

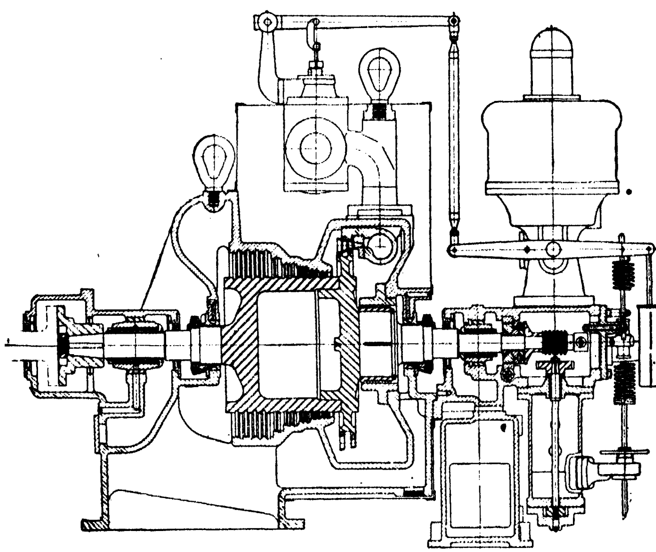


FIG. 217.—Westinghouse impulse-reaction turbine.

Types. For waterworks, all types are used with the exception of the complete reaction turbine, which on account of its nature is required to be of large size and power. In the impulse type, Fig. 217, expansion of steam takes place almost entirely in the stationary nozzles, and impinging of steam on the revolving buckets gives the driving torque. In the reaction turbine, Fig. 216, approximately one-half of the expansion in any stage takes place in the stationary buckets, imparting to the steam a velocity substantially equal to that of the moving buckets, so that it enters them without impact. Further expansion takes place in the moving buckets, the spaces between which gradually grow smaller from the inlet to the exit side, forming a ring of moving nozzles. Velocity imparted to the steam by expansion in the moving buckets produces a reactive effort on these buckets, which turns the rotor. This effect is similar to that produced by water issuing from a hose nozzle.

In the impulse-reaction turbine, steam is expanded in nozzles and discharged against a portion of the periphery of the impulse wheel. Intermediate and low-pressure stages are the same as in the ordinary reaction type. Substitution of the impulse element for the high-pressure section of reaction blading is made only in high-pressure stages of machines in which blades are short. The reason for this is that the clearance at the end of the unshrouded blading represents a relatively large ratio to the area through the blades, and it has been found impracticable to use shroud rings on buckets of small section. The unshrouded blades allow steam to spill over the ends, which, in short buckets, may be an excessive waste.

Figure 217 shows the Westinghouse impulse-reaction turbine. Steam is admitted from the governor to expanding nozzles which extend through a portion of the circumference. From these nozzles the steam passes through rotating buckets, then to stationary buckets, and again to rotating buckets, after which it enters a space extending entirely around the machine, from which it enters the first stage of the reaction blading. Expansion takes place through a sufficient number of stationary and moving buckets until final vacuum volume is attained. Labyrinth packing is used betwen the intermediate pressure and the vacuum to reduce steam pressure on the stuffing box at entrance end of machine. This principle is used in the manufacture of most turbines.

Merits. Small buildings; light foundations; light crane service; the only rubbing parts are in the bearings; simplicity of construction; great reliability; low cost of repairs, attendance, and operation; oiler attendant unnecessary; automatic oiling. A less efficient engineman may be employed, with consequent saving in wages.

Turbo-centrifugals vs. Reciprocating Pumps. Turbo-centrifugals require less floor space, less attendance, lighter foundations, and smaller buildings. They better utilize high vacuums and have less oil consumption. They are more adaptable to a greater range of steam pressures and vacuums. Under favorable steam conditions, turbo-centrifugals will equal or surpass in overall economy the vertical triple-expansion flywheel pumping engines.[7] The life of both is placed at 25 years by Waller, although some have claimed 10 to 20 years for turbines, and 30 to 40 for the flywheel pumps.[41a]

Cast steel should be used in all parts of valve bodies, steam chests, turbine cases, and wherever subjected to superheat, to withstand the pressures and temperatures.

Stuffing Boxes. Various metallic and fibrous packings, excellent for reciprocating rods, are not satisfactory for stuffing boxes on steam turbines. Packing in a steam turbine is subjected to a low pressure, the difference between pressure of the atmosphere and exhaust from the turbine, or some intermediate pressure slightly above atmosphere and atmospheric pressure. As a rule, the office of the packing is to prevent air leaking into the turbine, rather than steam escaping from it. Labyrinth packing, into which is admitted a jet of steam, or preferably water, of sufficient pressure and volume to exclude the air, is best practice.

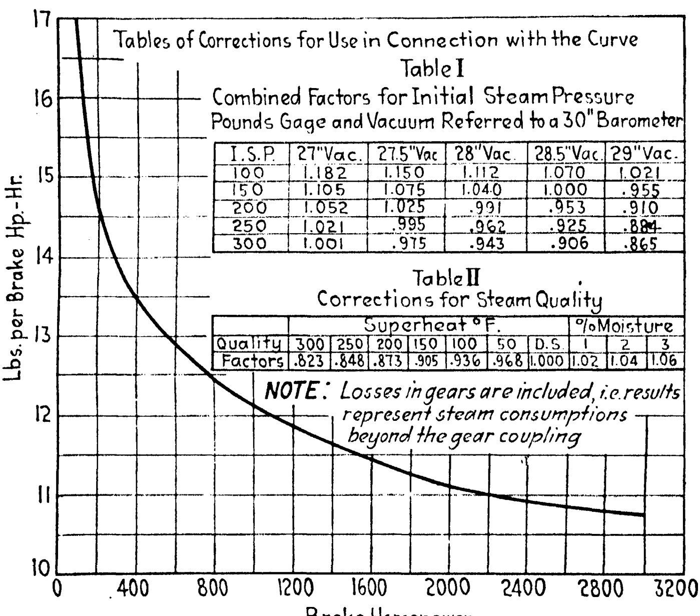
Economy. Efficiency of turbine will vary from 64 to 74 per cent., of gears 97.5 to 99, and of large pumps, 80 to 86. Duties of 100 million ft.-lb. per

100 lb. of steam were considered good in 1912; in 1922 the guaranteed duty had been raised to 200 million ft.-lb. for favorable operating conditions. Figure 218 is plotted to show the economies of present-day turbines (average). A 1200-hp. turbine designed for 50°F. of superheat, 27-in. vacuum, initial steam pressure of 200 lb. per sq. in., and 3 per cent. moisture will require:

$$11.86 \text{ (from curve)} \times 1.052 \text{ (upper table)} \times 0.968 \times 1.06 = 12.743 \text{ lb. of steam per brake hp. per hr.}$$

While a 1200-hp. steam turbine designed for 300°F. of superheat, 200-lb. initial steam pressure, 29-in. vacuum and 1 per cent. moisture will require

$$11.86 \times 0.823 \times 0.910 \times 1.02 = 9.061 \text{ lb.}$$



I.S.P.	27" Vac.	27.5" Vac	28" Vac.	28.5" Vac.	29" Vac.
100	1.182	1.150	1.112	1.070	1.021
150	1.105	1.075	1.040	1.000	.955
200	1.052	1.025	.991	.953	.910
250	1.021	.995	.962	.925	.884
300	1.001	.975	.943	.906	.865

	Superheat °F.							% Moisture		
Quality	300	250	200	150	100	50	D.S.	1	2	3
Factors	.823	.848	.873	.905	.936	.968	1.000	1.02	1.04	1.06

FIG. 218.—Relation of steam consumption and brake horsepower. (Turbine Equipment Co.)

There is marked economy, therefore, in increased pressure, temperature, and vacuum, and it is important to select values as high as possible.

The turbine[28] gains about 1 per cent. in economy for each 10°F. of superheat, and about 8 per cent. for an increase of vacuum from 28 to 29 in. mercury referred to a 30-in. barometer; whereas the reciprocating engine receives little, if any, benefit from increase of vacuum above 28 in. The turbine, also, can be coordinated with auxiliary units to better advantage to secure heat balancing and maximum plant heat economy, as by withdrawing steam from the intermediate stage of the turbine to heat boiler-feed water, or by utilizing to the fullest advantage in the turbine the steam exhausted from auxiliaries.

At present the maximum steam pressure used for waterworks steam-turbine units is about 300-lb. gage and the highest steam temperature 625°F. The pressure and temperature are not so high as those used in connection with large electrical generating plants, and it is, therefore, possible to assume that, in the future, waterworks engineers will consider higher pressures and temperatures than the ones given above. The condenser equipment should always be selected to give the highest economical vacuum for existing conditions.

The 30-m.g.d. centrifugal pumping unit of Mt. Royal, Baltimore,[43] developed on test 170 million ft.-lb. per 1000 lb. of steam, when delivering 45 m.g.d. against 180-ft. head, under a steam pressure of 172 lb., superheated 53°F., and a vacuum of 28.9 in.

With coal around $6 per ton, and average pumping of 10 m.g.d. and over, steam turbine will ordinarily (1923) show the best duty per dollar of annual charges.[7]

Table 147. Some Turbo-centrifugal Plants (1924)

Place		Delivery gals. per day	Ft. head
Pittsburgh, Pa.	1 unit	100,000,000	56
Cleveland, Ohio	1 unit	30,000,000	260
Cleveland, Ohio	2 units	100,000,000	50
		60,000,000	50
Omaha, Neb.	1 unit	30,000,000	90
Lynn, Mass.	15,000,000 combination pump	13,000,000	134
		2,000,000	244
Indianapolis, Ind.	1 unit	6,000,000	338
Philadelphia, Pa.	2 units	20,000,000	330
Philadelphia, Pa.	1 unit	25,000,000	279
Philadelphia, Pa.	1 unit	35,000,000	201
Toronto, Can.	2 units	24,000,000*	270
Toronto, Can.	1 unit	24,000,000*	60
Toronto, Can.	1 unit†	20,000,000*	270
Toronto, Can.	1 unit	10,000,000*	270
San Antonio, Tex.	2 units	6,500,000	273
St. Louis, Mo.	1 unit	20,000,000	.....
Youngstown, Ohio	1 unit	8,640,000	335
Youngstown, Ohio	3 units	8,640,000	350
New Haven Water Co.	1 unit	15,000,000	125
So. Pittsburgh Water Co.	1 unit‡	8,000,000	407.5
Baltimore, Md.	1 unit	30,000,000	190
Omaha, Neb.	1 unit	50,000,000	280
Chicago, Ill.	1 unit	75,000,000	165
Lansing, Mich.	2 units	20,000,000	200
Wilmington, Del.	1 unit	12,500,000	275
Wilmington, Del.	2 units	8,900,000	146

Turbines at Rock Island, Ill.; Atlantic City, N. J.; and Bay City, Mich. also.
* Imperial gal. (see p. 815).
† Guaranteed duty, 118 million ft.-lb.; duty during test over 130.
‡ Guaranteed duty, including auxiliaries, 100 million ft.-lb. per 1000 lb. steam at 120 lb.; no superheat; 28-in. vacuum.

MISCELLANEOUS TYPES

Non-pulsating pumping engines are used on wells. The water column never comes to rest as in other types of well pumps; this renders the pump practically free of pulsations, except those caused by the motor or gas-engine

drive. An efficiency of 80 per cent. is claimed for Luitwieler pumps by Luitwieler Engine Co., Rochester, N. Y. Pomona Pump, Union Iron Works, Inc., Kansas City, Mo., has two pistons in the well, the drive rod of one being inside the drive tube of the other. St. Johns, Mich., has one in operation; a few months' experience indicates saving over air-lift of over 3 tons of coal per month when pumping 235 g.p.m.

Pneumatic water-supply systems* supply water under pressure, eliminating elevated tanks (see Chapter 23), chiefly in small isolated installations, for rural homes and institutions. Water is pumped by hand or power into an airtight tank, where the air is compressed by the incoming water, to the pressure desired. Advantages are: tank may be out of sight within the building or buried; water is under better temperature conditions than in an elevated tank. An air compressor is generally included in a pneumatic installation to supply deficiency of air lost by absorption. Tanks have an output of 10 to 350 g.p.m., at pressures up to 115 lb. per sq. in. Tests at Purdue[32] on farm installations of capacities of 100 to 200 gal. per hr. involving a displacement pump and compression tanks showed overall efficiencies of 15 to 25 per cent., average, 17.5.

Pressure Tanks. Capacities range from 115 to 23,600 gal. Tanks range in size from 24-in. diam. and 5 ft. long to 120-in. diam. and 40 ft. long. J. B. Morrow, Oakland, Cal., publishes tables for finding contents of partly filled cylinders with both flat and bulged heads.[115] The largest tank that can be shipped complete is 10 ft. 6 in. in diam. by 40 ft. long.[116]

Hot-air engines have large size compared to steam engines, and low efficiency. They have wide use for small isolated plants. Donkin[33] gives fuel consumption in early British types as 1.8 to 4.2 lb. of coke per i.hp. per hr.

The *"Reeco" Rider*[34] *and "Reeco" Ericsson* hot-air pumping engines may be used in connection with pressure-tank systems as well as with elevated storage tanks. For shallow wells, where pump is attached to cylinder of engine, and for domestic use where daily consumption is small, and pressures from 25 to 75 lb., Ericsson type proves economical.

Hydraulic rams,† generally used where there is abundance of water but under low head, to lift a small quantity against a high head, operate by impulse imparted to the lifted water by a suddenly closed valve—the water-hammer principle. Chief disadvantage is noise of the continual pounding. Insertion of a piece of hose in the discharge line and provision of an additional air chamber is claimed by Kirchoffer to mitigate this nuisance.[35] Mechanically, the ram is the simplest, most efficient, most durable, and least expensive self-contained small pumping unit. Its chief field is for country homes, but it has been used for supplying railway tanks. The advantages over windmills are safety from storms, dependability, and no lubrication. (Oiling of windmills requires dangerous climbing of towers.)

It combines functions of motor and pump, and must be so considered in comparisons of efficiency. In 1895 D. W. Mead designed for West Dundee,

* Makers include Chicago Pump Co.; Kewanee Private Utilities Co.; Deming Co.; Lancaster Iron Works.

† Makers include Rife Automatic Engine Co., New York; Rumsey Pump Co., Seneca Falls, N. Y.; Hill Hydraulic Machinery Co., Seattle, and, Walworth Mfg. Co., New York. Ellsworth ram is described in *E. N.*, July 27, 1916, p. 164.

Ill., a ram with 43-ft. drive head and 2200 ft. of 10-in. cast-iron drive pipe, which is still operating successfully. S. B. Hill designed a waste valve which eliminates excessive hammering on seats, so that rams can act under higher heads. It is a balanced valve with downward discharge having 1-in. maxi-

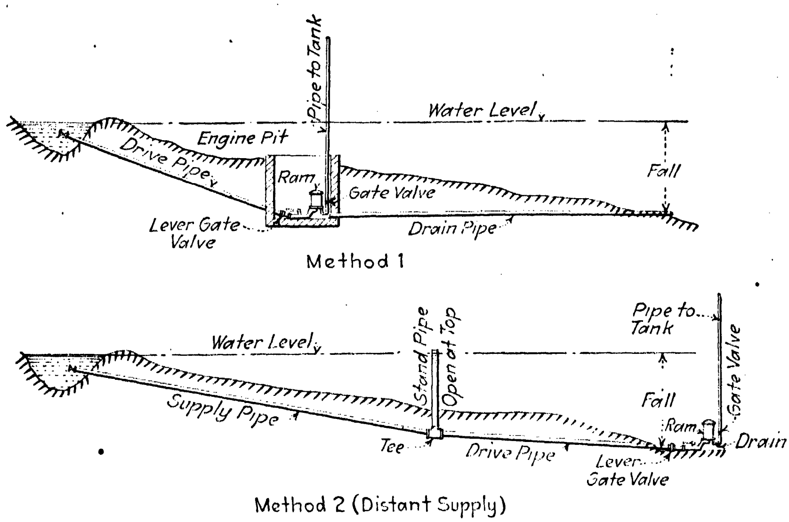


FIG. 219.—Two methods of securing fall for hydraulic rams.

mum movement. A Sterling 10-in. ram was tested at University of Washington; valve opening was $\frac{3}{4}$ in.; strokes per min. ranged from 44 to 97; quantity pumped, 0.1 to 0.8 cfs.; additional water used for power ("wasted"), 0.8 to 1.3 cfs.; supply head, 50 ft.; pumping head, 61 to 278 ft.; delivery head,

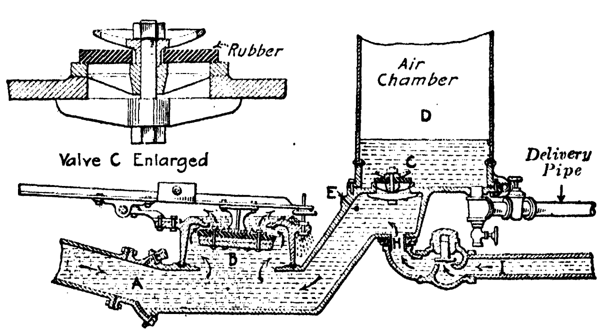


FIG. 220.—Cross-sectional elevation of hydraulic ram.[38]

111 to 327.5 ft.; average efficiency, by D'Aubisson's rule, 89 per cent.; by Rankine's, 85 per cent. No difficulty should be encountered in operating under supply heads of 1 to 100 ft. and delivery heads of 5 to 500 ft.[36] Rife[39] recommends use where supply water exceeds 3 g.p.m., and head 3 ft. Tests of 12-in. Hill ram in Maple Leaf Pumping Station, Seattle, under 48-ft. power

head and 131-ft. pumping head, gave efficiency, by D'Aubisson's formula, of 90.8 per cent., when delivering 0.56 cfs. and 82.3 per cent. for 1.08 cfs.[37]

Efficiency. D'Aubisson's formula for calculating efficiency is

$$E = \frac{Q_s(H + H_s)}{QH}.$$

Q is cfs. *flowing* in drive pipe; Q_s is cfs. flowing to standpipe through discharge* pipe; H is height (in ft.) from escape† valve to level of reservoir which feeds drive-pipe; H_s difference in level (in ft.) of water in supply reservoir and water in standpipe or other receiving vessel. Rankine's formula:

$$E = \frac{Q_s H_s}{(Q - Q_s)H}.$$

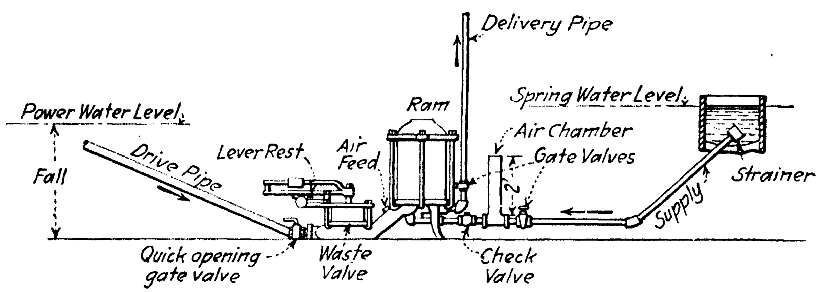


FIG. 221.—Arrangement of ram.

D'Aubisson's is correct, considering mechanism as receiving energy at one end and delivering it at other, while if machine is considered as elevating water only from one reservoir to other, Rankine's is correct.

Table 148. Data on Hydraulic Rams (Rife)[39]

Number	Dimensions: Height, ft.	Height, in.	Length, ft.	Length, in.	Width, ft.	Width, in.	Size of drive-pipe, in.	Size of delivery pipe, in.	Gals. per min. required to operate: Min.	Max.	Least fall recommended, ft.	Weight, lbs.
10	2	2	2	10	1	0	1¼	½	2	6	3	150
15	2	2	3		1	0	1½	¾	6	12	3	175
20	2	5	3	3	1	2	2	1	8	18	3	225
25	2	5	3	4	1	3	2½	1	12	28	3	250
30	2	7	3	7	1	3	3	1¼	20	40	3	275
40	3	7	4	9	1	8	4	2	30	75	4	600
60	4	8	6	0	2	3	6	3	75	150	4	1200
80	6	4	7	0	3	0	8	4	150	300	4	2200
120*	8	9	8	4	2	8	12	5	375	650	4	3000

* See example in Table 149. By using "Batteries" unlimited quantity can be delivered.

* Same as delivery pipe.
† Same as waste valve.

Table 149. Delivery of Hydraulic Rams, Gals. per Day

Power head, ft.	Pumping head, ft. 4	10	15	20	30	40	*50	60	70	80	90	100	120	140	160	180	200
2	540	192	128	96	64	43	29	24	....	....	....	....	....	....	....	....	....
3	....	301	192	144	96	72	58	43	37	27	24	....	....	....	....	....	....
4	....	432	256	192	128	96	77	64	55	43	38	29	24	....	....	....	....
5	....	540	345	240	160	120	96	80	69	60	53	43	30	26	....	....	....
6	....	....	432	302	192	144	115	96	82	72	64	57	43	31	27	24	....
7	....	....	505	378	235	168	134	112	96	84	75	67	50	36	31	28	25
8	....	....	....	432	270	192	154	128	110	96	86	77	64	55	43	38	29
9	....	....	....	485	300	216	173	144	124	108	96	86	72	62	54	43	39
*10	....	....	....	540	360	252	*192	160	137	120	107	96	80	68	60	53	43
12	....	....	....	....	430	301	230	192	165	144	128	115	96	82	72	64	57
14	....	....	....	....	505	353	270	224	192	168	150	135	112	96	84	75	67
16	....	....	....	....	....	432	323	257	220	192	171	154	128	110	96	85	77
18	....	....	....	....	....	486	390	303	247	216	192	173	144	124	108	96	86
20	....	....	....	....	....	540	430	336	288	240	214	192	160	137	120	107	96
22							475	370	303	264	235	212	176	151	132	118	105
24							520	405	346	288	256	230	192	164	144	128	115
26							....	470	375	328	278	250	208	178	156	139	125
28							....	505	430	354	300	269	224	192	168	149	134
30							....	540	465	405	336	288	240	206	180	160	144

* *EXAMPLE*
With supply of 700 gals. per min., 10 ft. fall, 50 ft. elevation, No. 120 engine will deliver 134,400 gals. per day:
700 × 192 = 134,400

* Factor = $\frac{\text{Power head} \times 40 \times 24}{\text{Pumping head}}$. Multiply factor opposite "power head" and under "pumping head" by number of gal. per min. *used* by engine and result will be number of gal. *delivered* per day. Efficiency is governed by ratio of fall to pumping head; 75 per cent. for ratio of 1 to 2½; 70 per cent. for ratio of 1 to 3; 66⅔ per cent. for ratio up to 1 to 18; 60 per cent. for ratio up to 1 to 23; 50 per cent. for ratio up to 1 to 30.

Windmills.* Horsepower of windmills of best construction is proportional to squares of diam. and cubes of velocities; for example, a 10-ft. mill in a 16-mi. breeze will develop 0.15 hp. at 65 r.p.m.; and with same breeze: 20-ft. mill, 40 rev., 1 hp.; 25-ft. mill, 35 rev., 1¾ hp.; 30-ft. mill, 28 rev., 3½ hp.; 40-ft. mill, 22 rev., 7½ hp.; 50-ft. mill, 18 rev., 12 hp.

Table 150. Capacities of Windmills, Gals. per Minute[40]

Designation of mill	Velocity of wind in miles per hr.	Revolutions of wheel per min.	Elevation, ft., raised 25 ft.	50 ft.	75 ft.	100 ft.	150 ft.	200 ft.	Equivalent actual useful hp. developed
8½	16	70–75	6.192	3.016	......	......	......	....	0.04
10	16	60–65	19.179	9.563	6.638	4.750	......	....	0.12
12	16	55–60	33.941	17.952	11.851	8.435	5.680	....	0.21
14	16	50–55	45.139	22.569	15.304	11.246	7.807	4.998	0.28
16	16	45–50	64.600	31.654	19.542	16.150	9.771	8.075	0.41
18	16	40–45	97.682	52.165	32.513	24.421	17.485	12.211	0.61
20	16	35–40	124.950	63.750	40.800	31.248	19.284	15.938	0.78
25	16	30–35	212.381	106.964	71.604	49.725	37.349	26.741	1.34

Increase in power from increased velocity of wind is equal to square of its proportional velocity; for example, the 25-ft. mill rated above for a 16-mi. wind will, with a 32-mi. wind, produce 4 × 1¾ = 7 hp. A windmill will run and produce work in a 4-mi. breeze (Table of wind velocities on p. 826). Marks[11b] gives theoretical hp. = $0.000005247\ D^2W^3$, where D is maximum diam. of wheel, in ft., and W is wind velocity, mi. per hr. Tests at Dodge, Kan.,[11b] indicated an efficiency of 14.4 per cent., for an Aermotor when $D = 16$ and $V = 20$.

* Among makers are U. S. Wind Engine & Pump Co.; Appleton Mfg. Co.; and Challenge Co. —all of Batavia, Ill.—A. J. Corcoran, Jersey City; Jackson Iron Works, San Francisco; Aermeter Co., Chicago.

STEAM BOILERS*

Boiler horsepower, according to standard adopted by A. S. M. E., is 34.5 lb. water evaporated per hr. from and at 212°F. This is equivalent to 33,479 B.t.u. per hr. In calculating the horsepower of steam boilers, allow (*a*) for tubular boilers, 15 sq. ft. of heating surface to 1 hp.; (*b*) for flue boilers, 12 sq. ft.; (*c*) for cylinder boilers, 10 sq. ft. Well-designed boilers, under successful operation, will evaporate from 7 to 10 lb. of water per lb. of first-class coal. To evaporate 1 cu. ft. of water requires the combustion of 7.5 lb. of ordinary coal, or about 1 lb. of coal to 1 gal. of water. Boiler horsepower = 0.069 gal. per min.[3] Each sq. ft. of heating surface evaporates 2.3 to 3.5 lb. of water per hr.; therefore, for an engine using 30 lb. of water per hp.-hr., each horsepower of the engine requires from 13 to 8.6 sq. ft. of heating surface in the boiler.

Factor of evaporation is ratio of heat required to generate 1 lb. of steam "from and at 212°F.," and $= \frac{(H - h)}{970.4}$, where H = total heat in 1 lb. of steam at boiler pressure and steam temp., and h = heat of liquid at feed temperature.

Table 151. Boiler Horsepower Required for Each Pump Horsepower, Counting 10 Sq. Ft. of Heating Surface per Boiler Horsepower[3]

Duty in ft.-lbs. per 1000 lbs. of dry steam	Boiler hp. per i. hp. of pump	Lbs. of steam per hr. per pump hp.†	Duty in ft.-lbs. per 1000 lbs. of dry steam	Boiler hp. per i. hp. of pump	Lbs. of steam per hr. per pump hp.	Duty in ft.-lbs. per 1000 lbs. of dry steam	Boiler hp. per i. hp. of pump	Lbs. of steam per hr. per pump hp.
40,000,000	1.63	49.5	120,000,000	0.55	16.5	165,000,000	0.40	12.0
50,000,000	1.32	39.6	125,000,000	0.52	15.8	170,000,000	0.39	11.6
60,000,000	1.10	33.0	130,000,000	0.51	15.2	175,000,000	0.38	11.3
70,000,000	0.94	28.4	135,000,000	0.49	14.7	180,000,000	0.37	11.0
80,000,000	0.83	24.7	140,000,000	0.47	14.1	185,000,000	0.36	10.7
90,000,000	0.74	22.0	145,000,000	0.46	13.6	190,000,000	0.35	10.4
100,000,000	0.66	19.8	150,000,000	0.44	13.2	195,000,000	0.34	10.0
110,000,000	0.60	18.0	155,000,000	0.43	12.8	200,000,000	0.33	9.9
115,000,000	0.57	17.2	160,000,000	0.41	12.4	....	......	......

† Calculated from formula 4, p. 472.

Table 151 is based on 1 sq. ft. of heating surface evaporating 3 lb. of water per hr., from 150° (temperature of feed), into steam at 150-lb. gage pressure. This is safe in most cases, but any desired increase may be made. For example: If 2½ lb. water per sq. ft. heating surface per hr. are all it would be safe to reckon on, then 20 per cent. added to boiler hp. of Table 151 would provide for such a case.

Combustion. The rate of combustion in a furnace is computed by the pounds of fuel consumed per sq. ft. of grate per hr. The size of coal must be reduced and the depth of fire increased directly, as the intensity of draft is increased. On 1 sq. ft. of grate can be burned an average from 10 to 12 lb. of hard coal, or 18 to 20 lb. soft coal, per hr. with natural draft. With forced draft nearly double these amounts can be burned.

* Manufacturers of boilers include Abendroth & Root Mfg. Co.; Babcock & Wilcox; Coatesville Boiler Works; Erie Boiler Works; Heine Boiler Co.; International Engineering Works; Parker Boiler Co.; Superheater Co.; Combustion Engineering Corp.

Boiling point of water at mean atmospheric pressure at sea level is 212° F. At an absolute pressure of 6 lb. per sq. in. (17.70 in. vacuum), this drops to 170.1° F.; while at 3 lb. absolute (23.83-in. vacuum) the boiling point is only 141.6° F.

Steam. Steam arising from water at boiling point (212° F.) has a pressure equal to the atmosphere (14.7 lb. at sea level). One cubic inch of water evaporated under ordinary atmospheric conditions is converted into 1 cu. ft. of steam. The specific gravity of steam at atmospheric pressure is 0.411 that of air at 34° F., and 0.0006 that of water at the same temperature; 27,222 cu. ft. of steam weigh 1 lb.; 13,817 cu. ft. of air weigh 1 lb. (both at atmospheric pressure).

Division of Heating Surface into Units.[3] First determine accurately the maximum number of pounds of steam that will be used. The maximum evaporation of a boiler is limited mainly by the quantity of coal which can be burned upon the grate. By dividing the total number of pounds of steam to be evaporated per hr. by 3, the total heating surface may be obtained with fair accuracy. The next step is the subdivision of this heating surface into a proper number of boilers. To evaporate a given quantity of water into steam, it is necessary to generate a certain amount of heat by combustion of fuel. The controlling factors are: kind of coal, area of grate surface, and draft. Ample grate surface is desirable. The kind of coal to be used should be determined before designing the boiler plant, the cost of various fuels available, and their calorific value, or relative evaporative power. With good coal, low in ash, approximately equal results may be obtained with large grate surface and light draft or with small grate and strong draft. Bituminous coal, low in ash, gives best results with high rates of combustion, provided ratio of grate surface to heating surface is properly proportioned. Coals high in ash require a comparatively large grate surface, particularly if the ash is easily fusible, tending to choke the grate. Where a strong draft is available, a smaller grate may be used than with moderate draft, as a thicker bed of fuel can be carried. When it is intended to burn low grades of fuel, provision for a large grate ought to be made in the first place. Then, if it is later desired to change to a fuel of a better grade, this can be done by reducing the size of the grate by bricking off a portion of it. If certain that there never will be a desire to use a poorer fuel, it would be uneconomical to provide larger grates than necessary.

Table 152 gives a very approximate estimate of the relative evaporative power of several kinds of coal, or the water that 1 lb. of coal will evaporate, the pounds of coal that may be economically burned per sq. ft. per hr., and the ratio of grate surface to heating surface with ordinary drafts.

Table 152. Evaporative Power of Coals[3]

	Pounds of water 1 lb. of coal will evaporate, with steam at 212° F.	Pounds of coal per sq. ft. of grate, per hr.	Ratio of heating to grate surface
Best bituminous	9.0	21	55
Ordinary bituminous	8.0	20	46
Anthracite (nut or larger)	8.0	13	32
Anthracite (buckwheat or rice)	7.0	12	26

[3] See also p. 512.

Selecting Boilers.[3] "Horsepower" is frequently used when estimating capacity of boilers, and buyers ask for boilers of a certain horsepower. This is an improper way to buy unless the heating surface per horsepower is clearly known. One bidder might offer a boiler with ample heating surface, another one with much less. Both boilers may develop the required horsepower, but the one with insufficient heating surface might do it only at increased cost for fuel. Boilers should not be bought with less than 1 sq. ft. of heating surface for every 3 lb. of water to be evaporated; in other words, with not less than 10 sq. ft. of heating surface per b.hp. It will usually be found good practice and economical to provide reserve power in the boilers and ample heating surface, particularly if the boiler plant is designed on the test duty of the engine, which is never realized in actual practice.

Firing Boiler. Coal of depth up to 12 in. is more effective than less depth. Admission of air above grate increases evaporative effect, but diminishes the rapidity of it. Air admitted at bridge wall effects a better result than when admitted at door, and when in small volumes, and in streams or currents, it arrests or prevents smoke. It may be admitted by an area of 4 sq. in. per sq. ft. of grate.

Boiler regulations for care and management have been issued since 1870 by Hartford Steam Boiler and Inspection Dept. and may be obtained on application. The A. S. M. E. has formulated a code (see *J.*, June, 1923, p. 368).

Stokers* have advantage of extracting from a given fuel a higher heat return than hand-fired furnaces. They can accommodate fluctuating boiler loads with a smaller variation of steam pressure than hand-fired furnaces; and produce less smoke; but they require coal and ash-handling apparatus, and add to upkeep cost of boiler.[11c]

Two chief types are chain-grate and forced-draft underfeed. Advantages[15] of former are: ability to take 100 per cent. overload; simplicity; ease of repair, operation by natural draft simplifies boiler-room equipment; efficient under light loads and overloads; first cost is half that of forced-draft type. Rules of operation have been established (1923) by committee of N. E. L. A. The forced-draft type has a larger unit capacity.

Superheaters† and Superheating. Installation of a superheater is equivalent to an increase in boiler capacity. As superheated steam prevents cylinder condensation, it will effect the greatest steam saving in the least economical engine.[15] A moderate degree of superheat is best suited to reciprocating pumping engines, and a higher degree to turbines.

Benefit of superheat[9c] in connection with engines of highest type is probably 6 per cent., as the upper limit of fuel economy that can be expected. Superheaters may be independently fired, or may be connected with the boiler. Engineers[3] are not agreed as to which arrangement gives the more economical returns. Requirements for a successful superheater are: security in operation, or minimum danger of overheating; economical use of heat applied; no exposure of joints to fire; provision for free expansion; disposition such that

* Among manufacturers of stokers might be mentioned: Babcock & Wilcox; CoKal Stoker Corp.; Westinghouse Elec. & Mfg. Co.; Vogt, Henry, Machine Co.; Combustion Engineering Corp.

† Among makers are: Babcock & Wilcox, Power Specialty Co., Superheater Co., all of New York.

joints can be cut out or repaired without interfering with operation of plant; case of application to existing plant. Independently fired superheaters have the following advantages: degree of superheat may be varied, independently of performance of boiler; can be placed at any desirable point; repairs can readily be made without shutting down the boilers. Some disadvantages are: separate firing and extra attention; extra piping; extra space; greater initial cost. Standard practice in this country tends toward the superheater contained within the boiler setting. In pumping stations 100°F. of superheat is sufficient. This corresponds to an increase of 8 to 10 per cent. in horsepower of boiler, insures practically dry steam at cut-off, and, therefore, produces a material reduction in losses due to cylinder condensation. At Chain of Rocks plant, St. Louis, 100°F. superheat resulted in a steam saving of 12 to 17 per cent. on triple-expansion and compound pumps, and a coal saving of 7.5 to 13.2 per cent.[15] This saving paid for the superheaters in 3.5 years. Predicted troubles with lubrication did not develop. Higher temperatures may interfere with lubrication,* and sometimes cause warping of valves. With 100°F. of superheat, and a steam pressure of 125 lb., producing a temperature of 425°F., no difficulties are ordinarily met. If superheat is used, metallic packing gives best results for piston rods and valve stems. Cast iron should be eliminated, as far as possible, from steam piping system, flanges should be wrought steel, fittings should be cast steel, and valves should be carefully selected.† If highly superheated steam is to be used, valves should be especially designed. The cost of superheater installation approximates $5 to $10 per hp., and material savings in the cost of piping may be made by the use of smaller piping on account of lower friction loss with superheated steam. Saving in fuel ranges from 7 to 15 per cent. The cost of repairs and maintenance is low, and many superheaters have been in service in pumping stations for upwards of 20 years with practically no maintenance cost.‡

Feed-water Heating.[3] Exhaust-steam feed-water heaters§ are used to heat water fed to boilers with steam exhausted by pumping engines and auxiliaries; saving is great. Generally speaking, for every 11°F. that feed water is warmed, there is a saving of 1 per cent. in fuel. With sufficient exhaust steam available, feed water at 50 to 60° can be raised to practically 200°, thus saving 12 per cent. of fuel. In pumping stations it is usually economical to use condensing apparatus in connection with the main pumps. If there is no station lighting plant, this leaves the exhaust steam from the auxiliaries only for feed-water heating, and, if feed water supply is cold, the heat contained in the exhaust from the auxiliaries is rarely sufficient to raise the feed water to more than 100 to 125°F. It is well to carry all condensation from jackets on main engines as well as from steam piping to a hot well. This will usually raise feed water with initial temperature of 60° to at least 100°, where steam pipe and jackets are effectively covered, and if it passes thence through the feed-water heater, the resulting temperature will be about 150°. One combination that works out favorably under conditions outlined is to

* Not found the case at St. Louis.[15]
† At St. Louis,[15] old cast-iron fittings stood up well 5 years under a total temperature of 500°F.
‡ Information from Power Specialty Co.
§ Among makers of heaters are Alberger Heater Co., Buffalo; Griscom Russell Co., New York; Cochrane Corp., Philadelphia; Bethlehem Shipbuilding Corp.

return the condensation to a hot well, carry the exhaust steam from independent auxiliaries to the feed-water heater, and extract sufficient heat from the flue gases, by means of an economizer, to return the water to the boilers at a temperature of from 200 to 225°F. Saving in coal effected is approximately 15 per cent., with initial feed-water temperature 60°F. Complete outfit—hot well, piping, feed-water heater, and economizer—may be covered by following approximate prices per hp. (furnished by Cochrane Corp.): 50 to 100 hp., $2.60; 150 to 250 hp., $1.65; 300 to 850 hp., $1.20; 1250 to 3000 hp., $.70. Provide thermometer wells and thermometers for obtaining temperatures resulting from various parts of the feed-water system.

Fuel Economizers.*[3] Fuel economizer is a long, narrow brick chamber between boilers and stack, or chimney, through which flue gases pass; it contains many vertical cast-iron tubes in which feed water circulates, entering at opposite end of chamber from gases. Factors to be considered before installing are: nature of auxiliaries and heat to be derived from them; methods of heating feed water, whether atmospheric heaters are used, and whether all, or part, of exhaust steam is used for heating; initial temperature of feed water and whether feed water is taken from hot well or from a cold supply; probable rise in temperature due to installation of economizer; cost of economizer; cost of additional building space; reduction of boiler heating surface made possible by economizer; extra cost of stack, or forced-draft apparatus necessary to compensate for loss of draft due to economizer; interest, depreciation, maintenance, operating cost, and insurance on all of those elements which result from installing economizer.

Air pumps and condensers† are required to pump cold water, hot water, and air. If duplex movement be used, while one piston is forcing water, the piston on opposite side may be encountering air or vapor, and meeting no resistance; consequently, that piston will race forward until it meets solid matter, resulting in throwing the steam valve on the other side too soon, thereby making the pump short-stroke. Where duplex air pumps and condensers are used, unless machine is so small that it must be pumping water steadily in order to provide enough to condense steam—and such are too small to be economical—it will be noticed that they do shorten their strokes, and thus impair their efficiency and economy. Single-acting air pumps and condensers are self-contained machines; each valve being thrown by the action of its own piston, it must complete its stroke in length whether the piston is moving in air, water, or vapor; consequently, nearer approach to theoretical quantity of water is obtained.

Jet condensers require much less water than surface condensers, but the water so used is ordinarily wasted. The speed of the pump and quantity of injection water can be regulated to suit the working conditions of the engine. It will quickly produce and maintain a high vacuum, removing nearly all the (atmospheric) back pressure from the piston of the engine. Being independent, it can be started and a vacuum formed before starting the engine. It will require less power than a connected air pump. As the power is taken directly from the boiler, the engine is relieved of that much load. It requires no

* Manufacturers include Alberger Heater Co., Buffalo; Power Specialty Co., N. Y.
† Among makers of condensers are: Allis-Chalmers Co.; Schutte & Koerting Co., Philadelphia; Westinghouse Co.

additional pump for the injection water; the air pump will lift the water from any point within the limit of suction.

Surface condensers should have about 2 sq. ft. of tube (cooling) surface per hp. for a compound steam engine, and 3 sq. ft. for turbines. Ordinary engines will require more surface according to the economy in use of steam. It is absolutely necessary to place air pumps below the condensers to get satisfactory results.

In ordering condensers give: (1) Diameter of steam cylinder and stroke of engine. (2) Revolutions per minute. (3) Steam pressure in boiler. (4) Maximum point of cut-off, or has the engine a plain slide valve? (5) Maximum temperature of the water, to be used for condensation, and is the water obtained from wells, stream, or pond? (6) Distances vertically and horizontally from the surface of the water supply to the floor on which the condenser can be placed. (7) Statement as to whether water is fresh or salt, clean, or dirty. (8) Indicator cards of the engine if possible.

Operating height of condenser above injection supply should be as little as possible; 18 or 20 ft. ought not to be exceeded. For high lifts of injection water, a charging valve and overhead supply will be found very advantageous in starting the condenser; this supply should be cut off as soon as vacuum is had. Start the condensing apparatus in advance of the engine, and shut down the engine before shutting down the condenser. It is estimated that 20 volumes of water absorb 1 volume of air; hence, if means were not taken to remove this air from the condenser, it would fill both condenser and cylinder, and destroy the vacuum.

The *effect* of a good condenser and air pump should be to make available about 10 lb. more mean effective pressure, with the same terminal pressure; or to give the same mean effective pressure with a correspondingly less terminal pressure. When the load on the engine requires 20 lb. mean effective pressure, the condenser does half the work; at 30 lb., one-third the work; at 40 lb., one-fourth, and so on. It is safe to assume that practically the condenser will save from one-fourth to one-third of the fuel, and can be applied to any engine, cut-off or throttling, where a sufficient supply of water is available.

The most economical *vacuum*, according to Marks,[11h] is 28 in. for steam turbines, and 26 in. for engines, with water of ordinary temperatures. A higher vacuum will not increase engine economies.

Table 153. Relation of Vacuum to Temperature Fahrenheit, of Feed Water

Vacuum, in.	Temp., degrees	Vacuum, in.	Temp., degrees
00	212	27½	112
11	190	28½	92
18	170	29	72
22½	150	29½	52
25	135	....	....

Condensing engines require from 20 to 30 gal. of water, at average low temperature, to condense steam represented by every gallon of water evaporated in boilers supplying engines—approximately for most engines from 1 to 1½ gal. condensing water per min. per indicated hp. With a limited supply,

water is used over and over, being cooled by circulation through a "cooling tower," or spray nozzles, and basin.

Instruments for recording all heat expenditures should be in every boiler room. See Toledo plant in *J. A. W. W. A.*, Vol. 10, 1923, p. 979.

FUELS

Weights of Fuels (for approximate computations). Petroleum, 6.5 lb. per U. S. gal., 42 gal. to the bbl.; crude oil, 8 lb. per gal.; gasoline, 5.6 lb. per gal.; 1 cu. ft. of anthracite piled loosely, 53 lb.; bituminous coal, 47 to 50 lb.

Relation to Pumping Costs. Coal in average pumping plant in 1920 constituted 40 per cent. of total pumping costs; as not more than 10 per cent. of the heat value is utilized, the most economical use of coal is desirable.[15] For Chicago costs, see Table 138, p. 474.

Cost of Fuel.[3] Steam consumption corresponding to various duties may be obtained from Table 141, p. 478. Annual cost of fuel $= \dfrac{S \times \text{i. hp.} \times T \times C}{E \times 2000}$.

S = lb. of steam used by engine per hp. per hr.
i hp. = average indicated horsepower developed.
T = number of hours during year pumping is required.
C = cost of coal in dollars per ton of 2000 lb.
E = evaporation factor of boiler, approximately 8 lb. of water evaporated per lb. of ordinary good bituminous coal.

As coal used bears some proportion to hours of pumping, coal saved by higher efficiency is necessarily reduced with shorter hours, reduction in operating cost is lessened, and there is smaller sum available to offset increased interest charges usual with high-duty machines. Stand-by losses reduce net efficiency materially, and, generally, it hardly pays to go into the most expensive forms of pumping machinery when the engine is to be run at a fraction of its rated capacity, or is to be operated only a portion of each day.[3]

Fuel Reserve Required. The Standard Schedule of the National Board of Fire Underwriters calls for coal-storage provision for 5-days supply minimum, and greater for long hauls. Gas supply shall be from two independent sources or from a duplicate gas-producer plant with a storage of at least 24-hr. supply. Oil supply shall be stored in underground tanks; at least 5-days, supply. Gasoline engines should have 36-hr. supply stored on premises, but 24-hr. storage has been tolerated.[46a]

Oil vs. Coal. Lucke[47] presents Fig. 222 for arriving at cost per million B.t.u. for comparing a coal-burning steam plant with an oil-burning steam plant or the oil-burning Diesel engine. At Wilmington, Del.,[86] records for 1922 indicated that oil at 4 cts. per gal. is more expensive than coal at $6.91 per ton. At Lynn, Mass., where oil was contracted for at price equivalent to $9 per ton for coal, two 175-hp. boilers in the main pumping stations were converted to oil for following reasons:* (1) oil is cheaper than coal; (2) it can be burned more efficiently than coal; (3) greater boiler capacity can be developed; (4) coal- and ash-handling charges will be eliminated; (5) variation in quality will be minimized; (6) banking of fires can be done very much more

* See also Langham, *Power*, Sept. 11, 1923, p. 423.

economically; (7) neater, cleaner, and otherwise better working conditions will be obtained.

One ton (2000 lb.) of coal is equivalent to 3.34 bbl. of oil at 325 lb. in practical heating value. For instance, if coal costs $5 per ton, equivalent fuel value will be derived from oil at $1.50 per bbl., or $1.66, if the labor saving in firing and handling ashes is reckoned at 10 per cent. Usual heat value of oil given is that determined in a bomb calorimeter; heat actually available in a boiler furnace is less, since all fuel oils contain a considerable percentage of hydrogen;

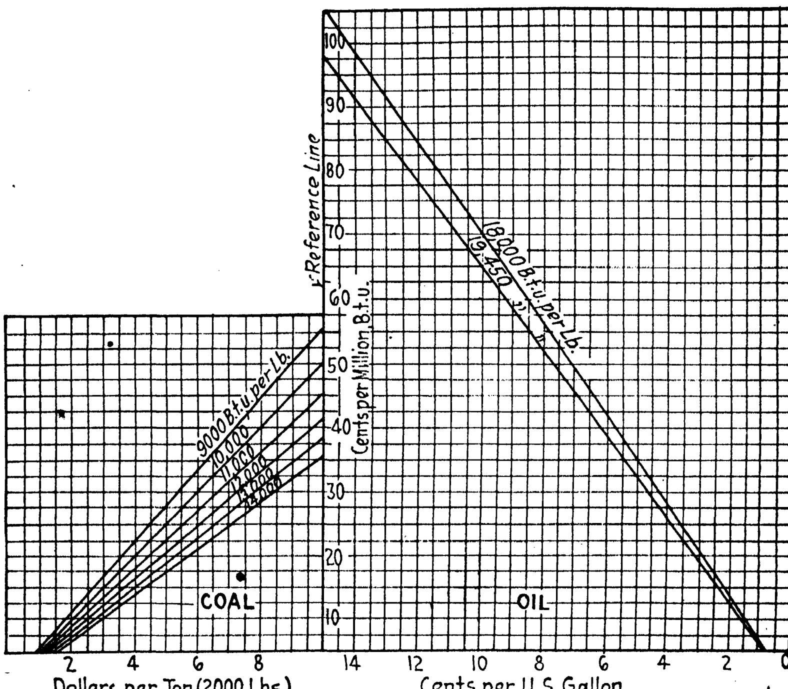


FIG. 222.—Fuel cost per million B.t.u. for coal and oil.[47]

the latent heat of the steam formed by combustion of this hydrogen wastes up the stack. Heavier grades of oil contain water in form of an emulsion, and it will not settle out. This lowers the heat value, but has no other disadvantage. When water settles out, as in light American crudes, it accumulates in tank and piping and goes over to the burners in a slug, putting out the flame. With emulsified oil a considerable amount of water can go through the burners with no bad effect. A small quantity of water in a heavy oil is an advantage; it helps to atomize the oil more thoroughly.[49]

Specifications for Coal.* (Abridged from New York City, 1915, "Standard Specifications for Furnishing, Delivering, Storing and Trimming Coal.")

Kinds and Sizes. "Anthracite" shall mean coal mined in the anthracite districts of Pennsylvania. "Semibituminous" shall be accepted in its usual commercial meaning. Not less than 90 per cent. shall pass over the mini-

* See also U. S. Government specifications, *Bull.* 116, Bureau of Mines, U. S. Dept. of Interior, 1916.

mum screen, and of broken, egg, stove, and chestnut, not less than 90 per cent. shall pass through the maximum screen.

Table 154. Sizes and Standard Analyses of Anthracite Coal
Tests with Stationary Screen Inclined 45°.

Names of sizes of coal	Shall pass through square-mesh screen of clear opening, in.	Shall pass over square-mesh screen of clear opening, in.	Per cent. moisture as delivered	Per cent. ash, dry coal	Per cent. volatile combustible, dry coal	Per cent. volatile sulphur, dry coal	B.t.u. per lb., dry coal
Broken....	4¼	2¾	4	11	8	1.5	13,200
Egg.......	3	2	4	11	8	1.5	13,200
Stove......	2½	1¼	4	12	8	1.5	13,000
Chestnut..	1½	¾	4	12	8	1.5	13,000
Pea........	¾	½	5	17	8	1.5	12,300
Buckwheat							
No. 1....	½	¼	6	18	8	1.5	12,200
No. 2....	¼	3/16	6	19	8	1.5	12,100
No. 3....	3/16	3/32	6	19	8	1.5	12,000

Table 155. Sizes and Standard Analyses of Semibituminous Coal

Names of sizes of coal	Through bars separated	Over bars separated	Per cent. moisture as delivered	Per cent. ash, dry coal	Per cent. volatile combustible, dry coal	Per cent. volatile sulphur, dry coal	B.t.u. per lb., dry coal
Run of mine.	Coal as it comes from mine, unscreened.	....	3.0	10	25	1.75	13,800
Lump.....	....	1¼ in.	2.5	9	25	1.5	14,000
Nut.......	1¼ in.	¾ in.	2.5	9	25	1.5	14,000
Slack......	¾ in.	....	3.0	11	25	1.75	13,600

Payment. Coal of quality superior to standard analyses shall be paid for as coal conforming to the standard, except that weight shall be corrected for moisture in excess of the specified percentage.

Method of Sampling. From deliveries exceeding 25 tons, but not exceeding 100 tons, a sample of 200 lb. shall be taken; for deliveries in excess of 100 tons, approximately one-tenth of 1 per cent. of the quantity delivered. The gross sample shall be broken by hand, or by crusher, to approximately pea size or smaller, and reduced by successive quarterings to not less than 5 lb. for laboratory tests. Samples for the determination of moisture content will be taken at the point of weighing and immediately collected in moisture-tight receptacles.

Method of Analysis. A portion of the sample shall be analyzed upon its receipt at the laboratory. Moisture, ash, volatile sulphur, and volatile combustible matter shall be determined by proximate analysis, and the heating value by an oxygen bomb calorimeter. The remainder shall be preserved for 30 days, in proper custody, after notification to the contractor of the results of the analysis, for a check analysis if required. The moisture

determined in the first analysis shall be final. If the first analysis shows a deficiency in the heat units or an excess of ash, volatile combustible, or volatile sulphur, and if the contractor questions the first analysis, he may, within 10 days after notification of the first analysis, request, in writing, a second analysis. This second analysis shall be final. If the aggregate deductions in gross weight computed on the check analysis equals or exceeds the deductions computed on the first analysis, the contractor shall pay for the check analysis. All deductions, except moisture, shall be based on the check analysis.

Correction of Gross Weight. (1) Moisture. If the moisture be in excess of the limits specified in the standard analysis, the gross weight of coal shall be corrected by an amount directly in proportion; *e.g.*, if the broken coal as delivered contains 6 per cent. moisture, a deduction of 2 per cent. of the gross weight shall be made. (2) Ash. The weight after correction for moisture shall be reduced at the rate of 1 per cent. for each per cent. of ash in excess of the standard analysis, computed to the nearest tenth of a per cent. (3) Deficiency in thermal units. The weight after correction for moisture shall be reduced at the rate of 1 per cent. for each 100 B.t.u. below the standard heating value, computed to the nearest 50 units. (4) Excess of volatile sulphur. The weight after correction for moisture shall be reduced at the rate of 5 per cent. for each 1 per cent. of volatile sulphur in excess of the standard analysis, computed to the nearest tenth of a per cent. (5) Excess of volatile combustible matter. The weight of coal after correction for moisture shall be reduced at the rate of 2 per cent. for each 1 per cent. of volatile combustible matter in excess of the standard, computed to the nearest tenth of a per cent. (6) Aggregate deductions. After the corrections for moisture all deductions above described shall be totalized, and deducted as a whole.

Excess clinker shall be cause for condemnation. All coal delivered, In addition to conformance with the standard analyses and other requirements, shall be required to show, after a test of reasonable duration, that it does not produce excessive clinker. When, in the opinion of the head of a department, the coal delivered from any mine or group of mines produces excessive clinker, delivery from such mine or mines, shall, on notification, be discontinued.

"*Mixed coal*" shall mean a mixture of different sizes or kinds of coal, the proportions of each size or kind being specified. The ingredients shall be paid for separately. The different kinds or sizes shall comply with the respective standard analyses.

Pulverized fuel* makes feasible the use of low-grade fuels. Combustion is completed while the fuel is in suspension in the air; proper preparation of the fuel as to fineness and dryness, an ample combustion chamber, and proper temperatures and adequate air supply are essentials. Great improvements have been affected in the process in the last decade.[50] The explosion hazard is practically negligible.[11d] Large fuel economics[95] are possible, as very high boiler efficiencies have been obtained. The reliability,[96] safety and efficiency of this method is well proven. At the Ford Plant, River Rouge, the installation proved superior to contemporary stoker installations; see also ref. 51.

*Material supplied mainly by Combustion Eng. Corp., New York. See also series of articles by Cole, Power, June, 1924, pp. 900, 940; Savage in Coal Trade Review, Sept., Oct., 1921; "Pulverised Fuel" by W. Francis Goodrich (Griffin, London, 1924).

CHIMNEYS

Functions of a chimney are to carry off the products of combustion, and to produce an adequate draft. Induced or forced draft is added where height would become abnormal for adequate draft.

Capacity.* Kent's rule: hp. = $3.33(A - 0.6\sqrt{A}) \times \sqrt{H}$.

Revised Kent rule: hp.† = $2.5D^2H^{\frac{1}{2}}$ (approx. for 1000 hp. and over); in which hp. = boiler horsepower; A = chimney area; H = chimney height; D = diam. Area required for oil fuel may be 60 per cent. of that for coal-fired boilers. Horizontal breechings should have area 20 per cent. greater than stacks.

Self-supporting steel smokestacks‡ may be had of any size; in all cases they should be lined for 30 or 40 ft. above the breeching connection with fire brick, and above that with common brick or cement plaster.§

Table 156. Chimney Height in Feet Necessary for Given Boiler Rating*

Height in feet	50	60	70	80	90	100	110	125	150	175	200
Diam. in inches	Commercial horsepower of boiler										
18	23	25	27	29	....	....	....	....	....	....	....
21	35	38	41	44	....	....	....	....	....	....	....
24	49	54	58	62	66	....	....	....	....	....	....
27	65	72	78	83	88	....	....	....	....	....	....
30	84	92	100	107	113	119	....	....	....	....	....
33	....	115	125	133	141	149	156	....	....	....	....
36	....	141	152	163	173	182	191	204	....	....	....
39	....	....	183	196	208	219	229	245	268	....	....
42	....	....	216	231	245	258	271	289	316	340	364
48	....	....	....	311	330	348	365	389	426	459	491
54	....	....	....	363	427	449	472	503	551	595	635
60	....	....	....	505	536	565	593	632	692	748	797
66	....	....	....	....	658	694	728	776	849	918	981
72	....	....	....	....	792	835	876	934	1023	1105	1181
78	....	....	....	....	....	995	1038	1107	1212	1310	1400
84	....	....	....	....	....	1163	1214	1294	1418	1531	1637
90	....	....	....	....	....	1344	1415	1496	1639	1770	1893
96	....	....	....	....	....	1537	1616	1720	1876	2027	2167
108	....	....	....	....	....	....	....	....	2392	2592	2770
120	....	....	....	....	....	....	....	....	2986	3226	3448

* Kent, based on 5 lb. of fuel per boiler hp. Height is measured from top of grate bars. With low-grade coal or mechanical stokers, increase hp. by 20 per cent.

The life of steel stacks is shorter than brick stacks. Duffy[91] claims that failure of stack where steel guy bands are placed is due to local rusting action. In San Francisco earthquake, 1906, individual guyed steel stacks suffered no material damage, affecting serviceability, while brick stacks were in most cases broken off.

* See also Cotton in *Mech. Eng.*, September, 1923, p. 531.

† See "Proportioning Chimneys on a Gas Basis," by Menzin, *J. A. S. M. E.*, Vol. 38, 1916, p. 31, and committee report in *J. Am. Soc. Heat & Vent. Eng.*, December 1923, p. 717. They advise use of gas basis.

‡ See Chapter 11*A* in Ketchum's "Structural Engineers Handbook," 3rd ed. (McGraw-Hill Book Company, Inc., 1924), and Adams in *E. N.*, July 20, 1905, p. 64.

§ Among makers of steel stacks are Blaw-Knox Co., Pittsburgh; Chicago Bridge & Iron Works, Graver Corp., Chicago; Littleford Bros., Cincinnati.

Table 157. Comparative Cost of Chimneys (1922)

Committee Report to Am. Ry. Assn., Proc., Division V, p. 389

Top diam. in.	Height ft.	Boiler hp.	Cost dollars	Cost per B.H.P. dollars	Annual maintenance and depreciation per cent.
		Steel (Self-supporting)*			
34	85	150	$ 569	$3.79	16
36	75	150	600	4.00	
38	90	200	758	3.79	
34	85	300	1138	3.80	
		Brick			
42	100	200	$2900	$14.50	3
Average.....	....	200	2750	13.75	
54	100	400	3100	7.75	
Average.....	....	400	2950	7.37	
66	100	600	3300	5.50	
Average.....	....	600	3150	5.25	
		Reinforced Concrete			
42	70	200	$1675	$8.37	2.5
Average.....	....	200	2492	12.46	
54	106	400	2450	6.12	
Average.....	....	400	2883	7.21	
66	100	600	3400	5.66	
Average.....	....	600	3233	5.38	

* A guyed steel stack 175 ft. high and 84 in. in diam. costs (1922), about 70 per cent. of a self-supporting steel stack.[11e]

Radial brick chimneys* are extensively used, being built of specially formed bricks by companies making a specialty of such construction; they also furnish designs. Chimneys of this type have proved very satisfactory.

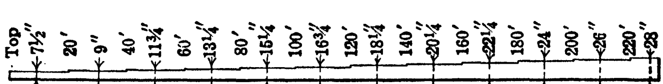


FIG. 223.—Heights of radial brick chimneys and corresponding thicknesses of walls.

Lining for Brick Chimneys. Height of lining for chimneys is found as follows: For ordinary conditions, where temperature does not exceed 800°F., the lining should be approximately one-fifth the height of the chimney; above 800 and below 1200°, one-half of the total height; above 1200 and below 2000°, full height.

Protection against Lightning.† For lightning protection two points are the minimum for any chimney of diameter up to 5 ft. Above this, one point

* Among makers of radial brick chimneys may be mentioned: American Chimney Corp.; Ballard, Sprague & Co.; Custodis Chimney Construction Co.; H. R. Heinicke, Inc.; Heine Chimney Co.; M. W. Kellog Co.; Rust Engineering Co.; Weiderholdt Construction Co.—all of New York City—and Summerhays & Sons, Rochester.

†From specifications of M. W. Kellogg Co., New York, *E. N.*, July 8, 1915, p. 77.

should be added for every 2 ft. in diameter or fraction thereof. The points of the conductor should be of copper ¾ in. in diam. by 8 ft. long, with a 1½-in. platinum-covered tip. They should be anchored at the top and extend from the bottom of the corbeling upward. The lower ends of the points are connected by a copper cable which encircles the chimney. From this loop a 1½-in. 7-strand No. 10 Stubs' gage copper cable is carried down the side of the chimney and connected to a copper ground plate of the three-winged type. The cable is anchored every 7 ft. with brass anchors, which support its weight.

Monoyer chimneys,[52] extensively used in Europe, have shafts made up of precast segmental concrete blocks, which interlock to form an octagonal section. Vertical reinforcing at each corner is cast in place after erection.

Table 158. Bottom Diameters of Radial-brick Chimneys, Ft.

Height of chimney, ft.	Internal diameter at top						
	3′	3′6″	4′	4′6″	5′	5′6″	6′
	Outside diameters in feet at bottom						
75	7.42	7.69	7.96	8.46	8.96	9.46	9.96
80	7.80	8.04	8.27	8.70	9.13	9.58	10.02
85	8.18	8.38	8.58	8.95	9.31	9.70	10.08
90	8.57	8.73	8.88	9.18	9.48	9.81	10.13
95	8.95	9.07	9.19	9.43	9.66	9.93	10.19
100	9.33	9.42	9.50	9.67	9.83	10.04	10.25
105	9.70	9.78	9.85	10.03	10.21	10.38	10.55
110	10.06	10.13	10.20	10.40	10.60	10.73	10.85
115	10.43	10.49	10.55	10.77	10.98	11.07	11.15
120	10.79	10.85	10.90	11.14	11.37	11.41	11.45
125	11.16	11.21	11.25	11.50	11.75	11.75	11.75
130	....	....	11.65	11.88	12.10	12.12	12.13
135	....	....	12.05	12.25	12.45	12.48	12.51
140	....	....	12.45	12.63	12.80	12.85	12.90
145	....	....	12.85	13.00	13.15	13.22	13.28
150	....	....	13.25	13.38	13.50	13.58	13.66
155	....	....	13.58	13.73	13.87	13.97	14.06
160	....	....	13.92	14.08	14.23	14.35	14.46
165	....	....	14.25	14.43	14.60	14.73	14.86
170	....	....	14.59	14.78	14.96	15.11	15.26
175	....	....	14.92	15.13	15.33	15.50	15.66

Reinforced-concrete* chimneys† have been successful since 1910; earlier chimneys caused some alarm from checks or cracks. In shape, they are generally coniform. It is claimed that they impose less foundation loads and occupy less space than brick chimneys and that they can be erected in less time than either steel or brick. Interior heat affects the shell because concrete is a poor conductor; the inside concrete for an inch or two is heated to a much greater degree than the exterior, and so tends to expand and crack the colder outside. This effect is most marked upon a thick wall. Concrete is an excellent fire-resisting material, although 1500°F. continued for 2 to 3 hr.

* Among makers of concrete chimneys may be mentioned General Concrete Construction Co.; Heine Chimney Co.; Rust Engineering Co.; Weber Chimney Co.; Wiederholdt Construction Co. —all with New York City offices—and Summerhays & Sons, Rochester.

† Based on text in "Concrete, Plain and Reinforced," Taylor and Thompson (Wiley, 1922).

will drive out the water of crystallization so as to take away the strength for a depth of ½ to 1 in. Lower temperatures affect it less, and tests at Watertown Arsenal indicate that a good cement mortar will not be appreciably injured at 600 to 700°F. The temperature in an ordinary chimney seldom exceeds 700°F. at the base; 400 to 500° is more usual. E. L. Ransome reports inner shell of a chimney 10 years old of concrete (cement, sand, and broken stone), in which he found the hottest part of the chimney opposite the flue perfectly sound and exceptionally hard. When temperatures above 750° are expected, it is safe to line with fire brick.

A reinforced-concrete chimney is so small at the base that it would topple over in a wind if not held by steel.* Thickness of outside shell must be sufficient to bear, with steel embedded, pressures due to weight and to wind. Vertical steel must be inserted all around to resist the pull caused by wind, and steel hoops placed at intervals to stiffen the vertical steel and prevent cracks due to difference in temperature between interior and exterior, and especially to resist vertical shear which corresponds to horizontal shear in a horizontal beam. Assume 50 lb. per sq. ft. of vertical surface as a maximum wind pressure, corresponding to 100 mi. per hr.; against a curved surface part of it is ineffective, so that the effective pressure against a chimney is not more than 33 lb. per sq. ft. against a vertical plane whose width is the diam. of the chimney. Concrete chimneys frequently crack at and above the top of the inner shell, so there is good reason for extending this shell higher than the usual one-third. It ought to be possible, however, so to lay and reinforce the concrete as to prevent cracking even with extremes of outside and inside heat. The inner shell must be entirely independent of the outer, as otherwise expansion caused by interior heat will tend to lift and crack the chimney at points of contact. If the inner shell does not extend to the top, the portion of the outer shell where it stops should be especially reinforced. Variation in thickness should be avoided, but if necessary, there should be additional reinforcement near the outer surface. Changes in section are bad because stresses are introduced which are indeterminate. Horizontal reinforcement should be spaced not more than 12 in. in the lower half of the stack. Steel must be carefully placed, vertical steel being preferably not far from the center of the wall and horizontal steel outside it. Where special temperature stresses occur, horizontal steel should be as near as practicable to the stressed surface, although not nearer than 1½ in. Steel must be well bonded, and concrete carefully placed and tamped around it. Outside surfaces must be formed by molds, no exterior plastering being permissible. Provide enough horizontal steel to take all vertical shear and to resist the tendency to expansion due to interior heat. Distribute horizontal steel by numerous small rods in preference to larger rods spaced farther apart. Foundations must be designed according to recognized engineering principles.† Wind causes vibrations which produce repetition of stresses. Following unit values are suggested: concrete, extreme fiber stress in compression, provided

* For analysis of stresses, see Hool & Johnson, "Concrete Engineers' Handbook," p. 816; *E. N. R.*, May 12, 1921, p. 803; for most complete treatise, see Taylor & Thompson, "Plain and Reinforced Concrete," 3rd ed., pp. 390, 660.

† For method of calculation, see "Concrete Engineers' Handbook," by Hool and Johnson, 1924, p. 557. Chimneys 160 ft. high are supported on steel roof girders at Montville, Conn. (see Contractor's Atlas, January, 1920).

it is capable of attaining strength of 2500 lb. per sq. in. in 28 days, 600 lb.; steel in tension, 14,000 lb.; concrete in shear, 60 lb. Concrete has proved suitable for the inner shell as well as the outer, but since mortar is easier to place in a thin wall there is no objection to it. Mild deformed steel gives greater adhesion, but round is safe. With T-shaped steel, perfect bond is more difficult because tamping into angles is troublesome. A radio tower, 672 ft. high, designed to withstand earthquakes* was built in Tokio in 1921.[53]

POWER, OTHER THAN STEAM

Utility. Until about 1915, steam was almost universally used for pumping engines; but electrical power, gas engines, and Diesel engines have come into use on small systems, and the steam-turbine-driven centrifugal pump (turbo-centrifugal) on large installations. Power pumps are connected to prime movers either directly, as in pumping engines, or by belt, chain, or gears. Motors can operate commonly at the same speed as centrifugal pumps, but must be geared down to accommodate the low r.p.m. of reciprocating pumps. Internal-combustion engines generally require speed-increasing (gears or belt) for centrifugal pumps, and speed-reduction for reciprocating pumps.

Belt drive requires consideration of size of belt and speed. Speed should, if possible, be kept below 4000 ft. per min. Width of single belt in in. = $\frac{600 \text{ hp.}}{\text{Belt speed, ft. per min.}}$. For double belt, substitute 400 for 600. An idler is required to reduce slipping where belt is at an angle with horizontal. An idler saves space but reduces efficiency. The drawbacks to belt drive are space required and the deterioration of an idle belt.

Mechanical Rules.[55] 1. Belt speed, per min. = diam. of pulley in in. × 3.1416 × r.p.m. ÷ 12.

2. Horsepower of belting, for open drives without idlers: (*a*) *Single Belts.* Belt speed, ft. per min. × width of belt in in. × 55† ÷ 33,000 = horsepower of single belt. (*b*) *For double and triple belts;* horsepower = one and a half and two times, respectively, that of single belt.

3. Width of belting in in. = horsepower × 33,000 ÷ belt speed in ft. per min., × 55† for single, 83 for double, or 110 for three-ply belts.

These rules apply where arc of contact, or angle of warp of belt around pulley, is 180°, *i.e.*, where diameters of belt pulleys are nearly equal; where the arc of contact is not 180° or approximately that angle, it is necessary to make an allowance in figuring the power which the belt will transmit.

Size and Speed of Pulleys and Gears.[56] To find *diam.*‡ *of driver:* Multiply diam. of driven by its r.p.m. and divide by r.p.m. of driver. To find *diam. of driven:* Multiply diam. of driver by its r.p.m. and divide by r.p.m. of driven. To find *r.p.m. of driver:* Multiply diam. of driven by its r.p.m. and divide by diam. of driven. To find r.p.m. of driven: Multiply diam. of driver by its r.p.m. and divide by diam. of driven.

Gear drive is fairly efficient, and requires little space, but is noisy and allows little flexibility.

* See also p. 828.

† With an arc of contact of 180°, 55 lb. per in. of width has proved to be a safe value for a single belt.

‡ If number of teeth in gear is used instead of diam. in calculations, substitute number of teeth wherever diam. occurs.

Table 159. Operating Costs of Prime Movers Compared[54]*

Type	Heat consumption per Br. H.P. hr., thousand B.t.u.	Overall thermal efficiency, per cent.	Superiority of Diesel engine†	Fuel	Aver. cost per 1,000,000 B.t.u., cents	Cost of 1,000,000 B.t.u. effective work, cents	Heat cost per one effective H.P. hr., cents
Steam engine: Non-condensing‡	40 – 28	6.3– 9.1	5.60–3.60	Coal	12	191–132	0.48–0.34
Condensing, superheated steam‡	28 – 16.5	9.1–15.4	3.60–2.30	Coal	12	132–78	0.34–0.20
Locomobile engine with superheated steam and reheater, condensing‡	17 – 15.2	14.9–16.7	2.40–2.10	Coal	12	81–72	0.21–0.18
Steam turbine, superheated steam: 200 to 2000 H.P.‡	24 –15.5	10.6–16.2	3.20–2.20	Coal and anthracite	{12 11	113–74 104–68	0.29–0.19 0.27–0.17
2000 to 10,000 H.P.‡	15.5–14.0	16.2–18.1	2.20–1.95	Coal and anthracite	{12 11	74–67 68–61	0.19–0.17 0.17–0.155
Gas engine: Without producer	10.4– 9.3	24.4–27.5	1.33–1.28	Natural gas, coke-oven or blast-furnace gas	{15 7	62–55 27	0.16–0.14 0.7
Suction§	14 –11.2	18.1–22.7	1.95–1.55	Anthracite	11	61–49	0.16–0.14
Diesel	8–7.2	32.0–35.3		Petroleum	15–18	56–43	0.14–0.11

* See similar tables, by Symonds, *J. N. E. W. W. A.*, Vol. 33, 1919, p. 168; by Muzzar, *J. N. E. W. W. A.*, Vol. 37, 1923, p. 245; by Creed, *J. N. E. W. W. A.*, Vol. 34, 1920, p. 16; by McMillin, *Power*, Dec., 18, 1923, p, 986 and *Manual of Am. W. W. Practice*, 1925, p. 379. † Factors to be applied to preceding column. ‡ Boiler losses are included. § Producer losses are included.

Chain drive requires less space than a belt. It is suitable for great power at slow speed. It has a longer life than a belt in damp or heated places,[6] and is quieter than gears if well designed and built, but under poor operating conditions may become noisy and inefficient.

Electric Drive. For reasonable economy, motors must run at relatively high speed and are, therefore, best adapted to high-speed pumps, such as centrifugals. They are also used for driving reciprocating and rotary pumps, through belting or other speed-reducing connections. National Board of Fire Underwriters requires with electric drive an auxiliary power, such as gas engines or Diesel engines, or an independent electrical supply source. If there is a distribution reservoir on the water system, the requirements of auxiliary power can be somewhat modified.

Advantages:[57] low operating expense—50 per cent. of that for steam, and lower than for internal-combustion engines; low first cost; compactness; simplicity; flexibility as to location; cleanliness; sturdiness; ease of control.

Disadvantages: lack of reliability; oversized motors increase power cost; corporation control of rates; variableness of voltage or frequency; dependence on exterior source of power; motors deteriorate when subjected to moisture, dirt, or neglect; steam turbines suffer less. High-voltage electrical equipment should be installed in dry places only.

Selection. Motor speeds (under load) in common use[19] are 1740, 1160, and 870 r.p.m. for 60 cycles, and 1450, 720, and 480 for 25 cycles. The capacity of a motor depends wholly upon how much heat it can stand; a 60-hp., 50° motor is about equivalent to a 50-hp., 40°. Motors rated on a 50° basis* should have a horsepower at least 120 per cent. of that required by the pumps; and on a 40° basis, 105 per cent. This margin is recommended by the Hydraulic Society for motors below 100 hp. to eliminate troubles from overheated motors due to deterioration or to room temperature.

D.C. Motors.[57] *D.c.* is obtainable only in large cities. Power from trolley circuits cannot be used on account of the variable voltage. The speed of a d.c. motor varies directly with the square root of the voltage, and the capacity of a centrifugal pump varies greatly with changes in pump speed. D.c. cannot be economically transmitted. Generator sets are installed sometimes to convert to a.c.

Constant-speed shunt and compound motors are suited to pumping. If pumps can be manually operated, a shunt motor is satisfactory, but where considerable power is required at starting, as with automatic control, compound d.c. motors should be used. D.c. motors are adjustable to speed, or can be designed for any speed. Approximately 10 to 50 per cent. change of speed can be obtained with standard d.c. motors, compound wound, and 15 to 100 per cent. with standard d.c. motors, shunt-wound. Standard adjustable speed motors are available up to 5 : 1 speed range. Usually, by selecting a pump having the right characteristics, ample variation in head and capacity can be obtained by field speed control alone.

A.C. Motors.[57] *Squirrel-cage induction motor* is simplest of all motors, as there are no wire connections to the rotating part. Constant speed, fixed by

*A 50°C. motor can heat up to 50°C. above room temperature without damage.

frequency and number of poles, admits of no adjustment. This is a rugged type of motor easily started either by hand or by remote control. Its operation demands little skill. Its principal requirements are that it be not overloaded for any appreciable time and that its bearings be kept oiled and free from grit. There are no moving rings or brushes to require attention. Though its speed decreases slightly with the load, the speed cannot be varied. Its starting torque is not large, and if used where even a moderately large starting torque is required, it draws a heavy current from the line. Its efficiency is high but its power factor is lower than is desirable, especially when lightly loaded, and this means larger transformers, lines, generators, etc., with a higher cost for the power used.[58] It is not well adapted to starting heavy loads.

The *wound-rotor* (*slip-ring*) *induction motor* has similar windings on the rotating and stationary parts, the ends of which windings are brought out to slip rings on the shaft. Resistors connected to the secondary circuit through the slip rings and cut in or out by a suitable controller vary the speed. Starting torque can be varied between 30 and 375 per cent. of full load torque. This permits starting the motor smoothly without drawing excessive currents from the line. The action is somewhat analogous to slipping a clutch to secure lower speeds and is, of course, wasteful. The variable-speed induction motor costs one and a half to two times as much as the ordinary squirrel-cage motor of same size, but saving in power resulting from its use, when the head varies considerably, will much more than pay interest and depreciation on additional investment.[59]

Both types of induction motors have the disadvantages: power factor always less than unity; extra losses in transmitting excitation to load; extra investment in generators, transformers, and power lines to carry the extra load. Their merit is in the very high starting torque available.[60]

Synchronous motors which permit of power and factor adjustment are favored with lower power rates. Small motor-generator sets must be installed to energize the field magnets. This type is seldom installed for less than 75 hp.[61] They are reliable in operation, but not so simple to start as induction motors. They have high efficiency but require some attention. Their electrical simplicity is comparable to that of the squirrel-cage induction motors. The speed is absolutely constant.[58] Two-hundred-horsepower synchronous motors are used at Camden, N. J.[61]

Motors for Reciprocating Pumps. Triplex pumps are commonly electric drive. Their low speed, 30 r.p.m., necessitates speed-reducing connections. The pump starts with a jerk which is severe on motor pinions and pump gears. Starting torque may vary from 125 to 250 per cent. of normal operating torque. Where the electrical utility has no regulations to contrary, squirrel-cage motors up to 50 hp. are used. The regulations of National Board of Fire Underwriters forbid squirrel-cage motors with both reciprocating and rotary pumps.* On larger sizes wound-rotor motors are used, as they do not draw so heavily on line current when starting.[62] Reciprocating pumps have been direct-connected to synchronous motors. It is generally possible

* For fire-protection pumps of private buildings, or small municipalities, see Proc. Nat. Fire Protection Assn., 1925, p. 487.

to place sufficient weight in the rotor to secure flywheel action. Full-sized bypasses must be provided to reduce the starting torque.[64] A recent type of motor, termed supersynchronous, gives an improved starting torque with higher power factor.[58]

Use of Motors with Rotary Pumps. The General Electric Co. advises use of self-starting squirrel-cage motors up to 50 hp., and slip-ring motors for larger sizes. The regulations of the local electrical utility may, and those of the National Board of Fire Underwriters* do, forbid use of squirrel-cage motors. Starting torque for rotary pumps is but 50 per cent. of normal, and no bypass is required.[58]

Use of Motors with Centrifugal Pumps.[58] The high speed of these pumps makes possible the selection of a motor of the same speed for direct connection. Starting torque of centrifugal pumps is low, and increases to normal at normal speed. When a pump is overloaded, the power requirements on the motor increase, and there is danger of overheating the motor. Many waterworks pumps are under changing loads due to variations in demands, with consequent changes in pressures. To proportion a motor for the normal conditions means that under maximum demand it will be overloaded, with the consequent danger of overheating. This menace can be met in several ways: (1) Proportion the motor for the maximum load; this prevents the motor overheating, but increases the operating costs, and gives a lower power factor. (2) Proportion the motor to normal conditions and throttle the discharge to keep the ratio of head and discharge constant. This method, although simple, is inefficient and should be utilized only when the condition of low head is infrequent. (3) Install a variable-speed motor, and vary speeds as required to keep power requirements within specified limits. The last is recommended.

With constant head, a squirrel-cage induction motor can be used up to 50 hp. If pumping against subnormal head is a frequent or prolonged occurrence, a variable-speed motor will effect a great saving in operation costs. If the motor is d.c., it should be of the compound-wound type.

Motor starters and controllers†[57] are essential to secure one of the chief advantages of electric drive: automatic starting and stopping, and control from a distance. Most small stations are equipped with automatic starters, operated either by drop of water level in the reservoir, or by pressure reduction in the system. A manual starter should also be provided as a convenience and a safeguard. Protective equipment should be provided to throw the starting switch in case of supply failure or overload. Motors above 3 hp. require special equipment to prevent damage to motor while coming up to speed. An auto-transformer is used in starting squirrel-cage motors. Remote control is particularly an advantage in railway water service.[63]

Manual starters should have no-voltage and overload devices to throw the starting switch in case of supply failure, or overload. Automatic equipment should also contain relays to open the circuit as required. Synchronous motors are usually controlled by compensators and field discharge switch. Automatic compensator may be actuated by push button, float switch, thermostat, or diaphragm pressure switch (pressure regulator).[81]

* For fire-protection pumps of private buildings, or small municipalities, see Proc. Nat. Fire Protection Assn., 1925, p. 487.

† See also committee report to A. R. E. A., *Bull.* 261, 1924, p. 174.

A *non-arcing float switch* does not carry the current delivered to the motor, but only controls a relay magnet or solenoid, which operates a switch controlling the motor circuit. Range between high and low water may be varied by adjusting the distance between the knots in the cord which operates the float switch. There is no necessity for carrying large wires from the float switch to the motor; an ordinary lamp cord is sufficient between the float switch and the self-starter; the float switch may then be installed near the tank and connected from any distance by means of lamp cord, or equivalent. Float switches are used to advantage where the area of the reservoir is large, and the change of elevation small.

Pressure regulator (gage type) is designed for use with the systems of automatically stopping and starting motor-driven pumps, working in connection with air or water systems, in which the governing is dependent on the pressure therein. The pressure regulator controls only the winding of a solenoid switch or relay and is a Bourdon gage with a special pointer provided with a non-oxidizable contact tip, which travels between two adjustable contact riders; the inner rider marks the low-pressure limit, and the outer rider the high-pressure limit. Circuit arrangements are such that but one side is connected to the gage at any time, so that danger of a short circuit is eliminated, and also the possibility of trouble from vibration. Range of adjustment is limited only by the range of the gage itself, and the contact riders may be set for starting and stopping the motor upon the variation of from 3 to 5 per cent. of the maximum pressure for which the gage is designed. This type should be used where the area of water storage is small and the standpipe is high, resulting in a large variation in the pressure, or in an installation where a float switch would be likely to get out of working order, by freezing or otherwise.

Rates must be carefully studied from the electric utility schedule. Electrical charges are based on demand requirements, (usually a percentage of the horsepower of the motors), and favorable rates are commonly obtainable if pumping is restricted to periods of low demand. At Concord, N. H., power charge was cut to 1⅓ cts. per kw.-hr., by agreement to pump between 8 p.m. and 6 a.m., except in case of emergency; 18 plants employing single-stage pumps in Iowa[6] in 1917, averaging 0.26 mgd. (0.040 to 1.000 mgd.) against 155-ft. head (14 to 305 ft.), required an average of 1080 kw.-hr. per million gal. (limits 500 to 1820); charges per kw.-hr. varied from 1.5 to 6 cts., averaging 3.58. Similarly, for two-stage pumping, six plants pumping from 0.025 to 0.135 mgd. (average 0.066) against heads of 150 to 525 ft. (average 300), required an average of 4390 kw.-hr. per million gal. (1160 to 9000); charges varying from 1.7 to 5 and averaging 3.15 cts. Power rates premised on the maximum amount used in any continuous 2-hr. period influenced the design of pumps at Cohoes.[85]

Studies by Purdue University indicate that power used in Indiana municipalities of 6000 to 11,000 people averaged 1310 kw.-hr. per 1,000,000 gal., or 9.0 kw.-hr. per mg. 1 ft. high. Average cost per mg. 1 ft. high ranged from $0.07 to $1.35, averaging 37 cts.[57]

Waterwheels are occasionally used to drive power pumps. At Peekskill, N. Y., waterwheels are used as alternative power for electricity for centrifugal pumps;[20] at Lewiston, Maine, two Worthington duplex pumps of 4-mgd.

capacity are driven from a single shaft through special gears by two 380-hp. Hercules turbines.[64]

Gasoline engines* (gas engines) have reached a high state of development through automotive use. They have the advantage of ease of operation and high speed (1000 to 1600 r.p.m.) controllable at the carbureter, making possible direct connection to centrifugal pumps through a flexible coupling. They weigh much less than oil engines of same capacity. Gas engines should have 25 per cent. excess capacity to allow for wear of valves.[67] The fuel cost is very high[65] and prohibits continuous operation. The gasoline consumption may be taken at 1 pt. per b. hp.-hr.,[66] although some manufacturers claim 0.7 to 0.9 pt. Oil consumption will vary from 0.05 to 0.07 lb. per b. hp.-hr. It takes $1\frac{1}{3}$ gal. denatured alcohol to do the same work as 1 gal. gasoline. Gasoline units are in general use as stand-bys for electric drive, generally on pumps below 4-mgd. capacity.[66] They are low in first cost, so that the stand-by charges are minimized. *S. A. E. formula for hp.*: B.hp. $= 0.4D^2N$, in which D is bore in in., and N = number of r.p.m. Power developed by gas-engine cylinders will lessen at high elevations above sea level, in certain proportion to diam. of cylinder, on account of diminished atmospheric pressure and consequent diminished quantity of oxygen per unit of air. For example: Gas engines of 100 b. hp. are scaled down to 80 hp., with fixed size of cylinder, under guarantees made for Denver, as against operation of same engine at or near sea level.

Gas vs. Steam. Small steam pumping plants are extravagant of fuel, while small gas plants are practically as economical as larger, and although application of steam power to pumping is more direct than with gas, great fuel economy of gas-power apparatus enables it to operate at a profit at a much lower mechanical efficiency than the steam engine. For example: A small gas engine geared to a triplex power pump may result in a total mechanical efficiency of 75 per cent.; then, if engine and producer give 1 b. hp. for $1\frac{1}{2}$ lb. of coal per hr., coal per pump-hp. will be 2 lb. Small steam pumping plant with a mechanical efficiency of 94 per cent. requires not less than 4 lb. of coal per

Table 160. Consumption of Gasoline for Pumping

Gasoline or distillate per effective horsepower per hr., gal.	Approximate consumption of gasoline or distillate in gal. per 24 hrs. when pumping 100 gals. per min., 100 ft. head				
	Total efficiency of pump and drive				
	30 per cent.	40 per cent.	50 per cent.	60 per cent.	70 per cent.
$\frac{1}{10}$	20.2	15.2	12.1	10.0	8.6
$\frac{1}{9}$	22.5	16.8	13.5	11.3	9.6
$\frac{1}{8}$	25.3	18.9	15.2	12.8	10.8
$\frac{1}{7}$	28.9	21.8	17.3	14.4	12.4
$\frac{1}{6}$	33.7	25.3	20.2	16.7	14.4
$\frac{1}{5}$	40.4	30.3	24.3	20.0	17.3

To find approximate number of gallons of gasoline or distillate required per 24 hr. when pumping 600 to 1200 gal. per min., multiply water horsepower by 0.027, or when pumping 100 to 500 gal. per min., multiply by 0.032. For theoretical water horsepower, multiply gal. per min. by total head in ft., including friction in pipes, and divide by 4000. Error = plus 1.1 per cent. 100 g.p.m. = 0.144 m.g.d.

* The reader is referred to "Internal Combustion Engines," by Robert L. Streeter (McGraw-Hill Book Company, Inc., 1923). Also "Gasoline Engine Drive for Centrifugal Pumps," by T. M. Heermans, *Power*, Mar. 8, 1921, p. 376.

pump-hp.-hr., and is doing well to accomplish this. Banking coal fires is costly, but is necessary to assure stand-by service for steam pumps. Stand-by gas engines entail no fuel charge when idle.

Starting gasoline engines requires starters, which should be operated in large engines by electricity; if gas engines are a stand-by to electric drive, storage batteries must be used. At St. Thomas, Ont.,[69] these batteries were charged from the engines, but infrequent operation resulted in insufficient charging, and the city current was utilized by means of a tungar rectifier; all batteries were eventually assembled conveniently behind the switchboard. Each engine should have a separate gasoline tank.

Gas engines* supplanted steam engines under steam from boilers fueled by natural gas at a saving in fuel (natural gas from city-owned wells) of about two-thirds, at Clarksburg, W. Va.[70] Equipment consists of 4-mgd. centrifugal pump for 35-ft. head., driven by 50-hp. engine (filter-bed service), and 4-mgd. pump, for 350-ft. head, driven by 350-hp. engine. Double-helical gears step up engine speed from 200 to 1385 r.p.m. During 10-day continuous run of both units, 82,200 cu. ft. of gas per day were consumed; delivery was held up to 4.1 mgd. against 30- and 330-ft. heads, giving load of 285.5 water hp. At 6 cts. per 1000 cu. ft. (used as basis for reckoning, although local commercial rate is 8 cts.), fuel costs $6.96 per hp., per year, or 0.079 ct. per hp.-hr. Cost of equipment, foundations, cranes, piping, etc. was near $31,400. Interest, depreciation, and up-keep, all covered by 10 per cent., would amount to $3140, giving pumping cost per hp.-hr., exclusive of labor and supplies, of 0.217 ct.

Gas producers† have proved economical where no other gas supply is available. At Manchester, Mass.,[71] the units showed an average efficiency of 150 million ft.-lb. per 100 lb. of fuel. Town gas is seldom used in Great Britain, producers being the rule. Johnstone-Taylor[87] says that well-designed gas engines and producers will give 1 b. hp. on less than 0.75 lb. good anthracite.

Oil engines‡ have been used in water works practice to drive both reciprocating and centrifugal pumps. The expiration of the Diesel patents in the United States in 1913 gave a great impetus to American makes,§ which have now been developed to a point where they are economical and dependable. They are made up to 6000 hp. At New Britain, Conn., Hazen in 1923 geared a horizontal duplex plunger pump to a two-cylinder, two-cycle, vertical, solid-injection full-Diesel engine. Distinctive features of the Diesel system are the use of a compression temperature sufficient for ignition and the atomization of the fuel oil by compressed air. The Diesel engine is unique among internal-

* Some types of engines having a proper manifold, notably Buda, operate also with gas or kerosene.

† Makers of producers include Coatesville Boiler Works; Milwaukee Reliance Boiler Works; Smith Gas Engineering Co., Dayton, Ohio.

‡ Reader is referred to: "Diesel Engines," by Lacey H. Morrison (McGraw-Hill Book Company, Inc., 1923). "Internal Combustion Engines," by Robert L. Streeter (McGraw-Hill Book Company, Inc., 1923). "The Design and Construction of Oil Engines," by A. H. Goldingham (Spon & Chamberlain, 1922). "Diesel Engine Practice," by A. H. Pritchard, *Power Plant Eng.*, Sept. 15, 1921, p. 901; Oct. 1, 1921, p. 952. "Diesel Oil Engine for Water Works Service" by Lucke, J. N. E. W. W. A., Vol. 37, 1923, p. 145.

§ Makers of Diesels include:[75] Allis Chalmers, Busch-Sulzer Co., Ingersoll-Rand Co., McIntosh & Seymour Co., William Graff & Sons, New York Ship Building Co., Worthington Pump & Machinery Corp.

Makers of semi-Diesels include: August Mietz Corp., Fairbanks-Morse Corp., Bessemer Mfg. Co., Burnoil Engine Co., St. Mary's Engine Co.

Makers of heavy-oil engines include: Bolinders Co., New York.

combustion engines, in that the admission of the fuel is gradual, so that the combustion is not of an explosive nature, and during combustion the pressure does not rise appreciably.[47]

There are two types: Diesel and semi-Diesel. The latter differ from the true Diesels in having an uncooled portion of the combustion space; they are subject to lower pressure.[11f] Semi-Diesels do not cost so much as Diesels; they run at lower pressures, have fewer working parts, but have lower efficiencies and are not so convenient to start.[46b] More semi-Diesels than Diesels were built in the United States in 1921.[72]

Advantages. (1) There is high thermal efficiency. An oil-engine reciprocating pump unit can operate on a fuel cost equivalent to a duty of 225 million ft.-lb. per 1000 lb. of steam.[73] (2) Efficiency is independent of size of unit. (3) Internal-combustion engines are self-contained plants, involving no costs for auxiliaries and boilers. (4) Variable loads can be accommodated by using several small units, as loads can be taken without any preliminary warming up. (5) Operating cost is lower than gas or steam.* (6) Charges for maintaining steam in idle stand-by equipment are absent. (7) A gas engine can be put into service to take additional load more quickly than a steam engine. (8) Fuel consumption is practically constant under a wide range of load; loads 30 to 40 per cent. above the rated capacity may be carried.[11g] (9) Diesel engines require about one-third the lubricating oil of gasoline or semi-Diesel engines.[46c] Proctor[74] prefers Diesels to gas engines for stand-by service. Diesels are used as stand-bys in many hydro-electric plants in Europe (for list, see *Elec. World*, Jan. 8, 1921, p. 95). Symonds found them ideal installation in pumping stations requiring 25 to 150 hp.[71]

Disadvantages. (1) Gearing is necessary in driving most pumps. (2) Heavier construction than gas engines is important in export shipment and floor loads. (3) Large floor space is required. (4) Compared to pumping engines, they are not so easily adapted to positive displacement pumps,[76] and they are not so reliable. (5) The first cost is high. (6) Sticking of exhaust valves caused considerable difficulty at Palo Alto.[77] (7) Liability to smoke when subjected to fluctuating loads. Lucke[47] claims that well-designed air-injection Diesel engines are smokeless. (8) They operate at constant speed. (9) Lack of accessibility to moving parts in some types adds to cost of repairs.[78] (10) Complaints are made in residential districts of vibrations, noises,† and odors.[46d] Muffling will lessen these at a small loss of power. (11) Remote control is not possible for starting, although they can be stopped from a distance. (12) There is diminution of power at high altitudes (see Robie, *Eng. Mining*, June 5, 1920, p. 1269). (13) There is uncertain life, although McMullin[94] places it at 25 yrs.

Fuel. Diesel engines can use any fuel oil that can be forced through a pipe. Tests of waterworks pumps at Amsterdam, Holland,[78] with an exceedingly heavy Mexican crude oil, sp. gr. at 15°C. = 0.862, with an excessive percentage of sulphur and asphaltum, proved that it could be used successfully, but only under the most favorable conditions. Commonly, Diesel

* For comparison of steam-turbine and Diesel operating costs, see *Power*, Nov. 16, 1920, p. 787. At Sioux Falls,[94] two Diesels driving triplex pumps replaced one vertical triple-expansion and one horizontal cross-compound steam pump in 1913 and cut cost of fuel, lubricants, waste, and repairs from $20.48 to $10.61 per mg.

† Noises are analyzed in *Eng.*, June 11, 1920, p. 797.

engines require ½ pt. (0.0625 gal.)[74] of crude oil per hp. per hr. Tests at Palo Alto, Cal.,[77] gave at full load, 0.0492; at three-quarters load, 0.0487, and at half load, 0.0556 gal. per b. hp. Engines for Gloucester, N. J.,[79] were purchased under a guaranteed consumption of 0.065 gal. (0.48 lb.) per b. hp.-hr.; in operation they averaged 0.0624 gal., on basis of 18,500 B.t.u. per lb. of fuel oil. Every precaution should be taken to eliminate foreign matter from the fuel oil. Quality of fuel oil is most important;[79] Moore[82] cites eight desirable tests: specific gravity, closed flash point, cold test, heat value, ash content, water content, coke value, and content of hard asphaltum.

Gearing. Since oil engines operate at constant speed, gearing is essential in most cases for connecting to centrifugal or reciprocating pumps. Speed of Diesel engines varies from 150 to 375 r.p.m.[47] The modern herringbone gear can be used in single reduction for speed ratio of 1:5½ bringing the resultant speed within the range of centrifugal pump speeds. For low-heads it may be possible to use a low-speed centrifugal direct connected.[47] A reciprocating power pump with the water end equipped with an outside center-packed plunger, operating normally at 165 r.p.m., was run at 200 r.p.m. successfully at Buffalo.[47] Single-reduction gears met requirements of rim speed. A triplex pump usually operates at a speed of 35 to 60 r.p.m. To obviate the gear losses, Diesel engines in the pumping station at Gloucester, N. J., are direct connected to generators at 400 r.p.m., which supply current for electric drive of the centrifugal pumps.[79]

STATIONS*

Superstructures.† Fireproof construction is essential. Brick and stone are most satisfactory materials for exterior, and terra cotta, enameled brick, or other material presenting a clean and non-absorbent surface, for interior. Concrete floors should be surfaced with terrazo or tile; where economy prevents, concrete surfaces should be covered with a heavy non-absorbent paint adapted to concrete. A double ceiling is desirable to prevent sweating; rock plaster on metal lath hung from lower-chord purlins will suffice. Booster pumping stations in residential districts should be made as soundproof as possible and architecture should conform to near-by structures.

Architectural Details. Copper roofs have relatively short life in some situations, due to acid in atmosphere and other causes. Red Conossera Spanish and similar tile roofs for gate houses are satisfactory in appearance; but for weathertightness in climate with high winds and drifting snow they are not adapted, unless laid on steep slopes with tight roofs beneath. Thin slates and terra-cotta tiles are liable to breakage by stones thrown by mischievous persons, by bullets, and sometimes by frost. Board of Water Supply, New York, has adopted strong reinforced-concrete tiles to meet this trouble and for other reasons.‡ These tiles are about 24 by 32 in., 1⅛ in. thick. Roof frames are of steel protected from corrosion by heavy mortar coat. In some waterworks structures wood in roofs and other parts is rapidly decayed

* For operation rules for army cantonments, see *J. A. W. W. A.*, Vol. 6, 1919, p. 174.
† See "Modern Waterworks Pumping Stations," by C. B. Burdick, *J. A. W. W. A.*, Vol. 11, 1924, p. 371, which shows exterior designs.
‡ Manufactured by American Cement Tile Mfg. Co., Pittsburgh.

by dampness. Ample ventilation is a help. Windows should be of heavy wire glass, with Monel metal or other rustless wire,* and in some situations must be protected by shutters, grilles, or strong wire netting. For very permanent construction set glass in metal sash or directly in the masonry. Strong rustless wire screens of coarse mesh are useful in chimney tops to keep out birds and other obstructions; also outside of windows to keep out birds and prevent breaking of glass by stone throwing. Doors should have sills an inch or more above floor. In some buildings freezing of condensed moisture on inside of building interferes with operation of doors, windows, and other moving parts. Doors should be at least $2\frac{1}{2}$ in. thick.

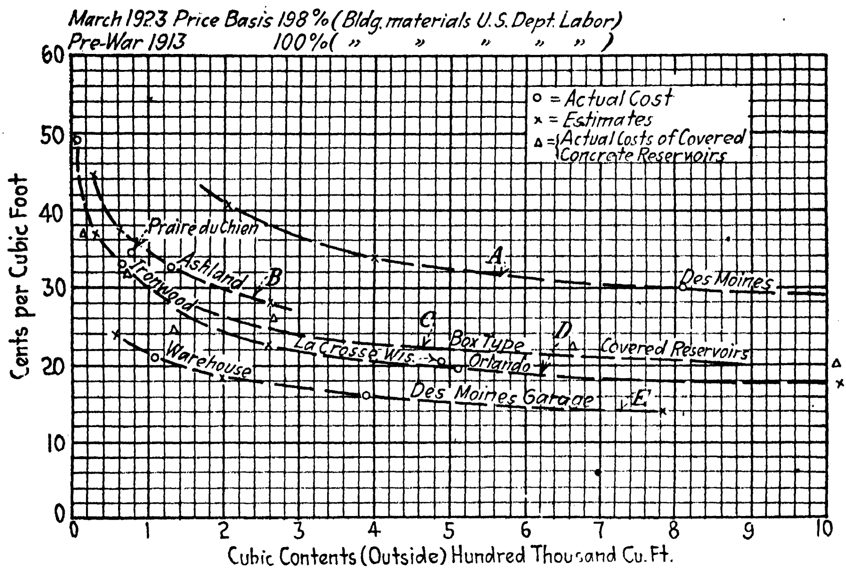


FIG. 224.—Approximate cost of stations and reservoirs reduced to 1923 price basis.[80] *A*, *B*, *D*, pumping stations; *C*, covered reservoirs; *E*, auxiliary buildings.

Architectural treatment† of gate houses and pumping stations has been found an asset in impressing the consumer with the care taken with the water supply. For Catskill aqueduct houses, see *E. R.*, Feb. 5, 1916, p. 174, and *Proc. Concrete Inst.*, 1915, p. 563.

Suction wells are often placed low enough so that water from the intake enters by gravity. This generally means that they are sunk low in wet ground, and that the large size makes the sinking a real problem in foundations. Commonly construction is sublet to one of the large foundation companies specializing in such work. For Louisville work, see *E. N. R.*, May 9, 1918, p. 905; for Vicksburg, see *E. N. R.*, Dec. 31, 1925, p. 1078.

* On Catskill aqueduct structures, such glass was furnished by Pennsylvania Wire Glass Co., Philadelphia.

† For dams, see p. 118.

Table 161. Cost of Pumping Station Buildings[80]

	Cost	Contract date	Cubic feet (thousands)*	Cost per cubic foot, cents	Cost price base†	Present price base (March 1923)‡	Present cost per cubic foot,‡ cents
Pumping stations							
Des Moines, Iowa	$220,479	1920 23	810 0	27 0	178	198	30 0
Ashland, Ky	34,194	Oct 1921	130 0	26 4	159	198	32 8
Orlando, Fla	91,800	Sept. 1922	510 0	18 0	180	198	19 8
Ironwood, Mich :							
Main station	29,155	July 1920	65 0	44 8	269	198	33 0
Sub-stations	9,644	July 1920	7 5	64 2	269	198	47 6
Manistique, Mich	71,379	Sept 1921	140 0	51 0	156	198	65 0
La Crosse, Wis	50,834	Nov 1912	484 0	10 5	100	198	20 8
Prairie du Chien, Wis	21,017	Sept 1921	77 8	27 0	156	198	34 3
Auxiliary buildings							
Des Moines garage	51,027	May 1922	390 0	13 1	160	198	16 2
Des Moines warehouse	18,754	May 1922	111 0	16 8	160	198	21 0
Des Moines cottages							
3 cottages, 4 families, 22 rooms	20,430	May 1922	18 0	29 2	160	198	36 0
Ironwood cottages							
2 cottages, 2 families, 12 rooms	16,900	July 1920	16 0	44 5	269	198	33 0

* Each building, where more than one Contents based on outside measurements footing to average roof

† Building materials U S Department of Labor

‡ As of date Apr 1, 1923

Safe Station Duty. The following facts are to be considered in arriving at safe station duty: (*a*) Capacity and head. (*b*) Boilers. Two boilers per battery are used in the pumping stations of New York City. (*c*) Method of firing. (*d*) Grade of coal. (*e*) Pump efficiency. (*f*) Carefulness of maintenance and operation. (*g*) Efficiency of attendants and the general standards of the department. Assume 115 million ft.-lb. duty per 100 lb. of coal for large stations, for approximate estimates.

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Vol. 11, 1924, p. 296. **52.** *E. N. R.*, July 12, 1917, p. 75. **53.** *E. N. R.*, May 19, 1921, p. 847. **54.** *Bull.* 156, Bureau of Mines, U. S. Dept. Int., 1918. **55.** Belting Manual, Graton & Knight Mfg. Co. **56.** *Bull.* 112, Goulds Mfg. Co. **57.** *Bull.* 4, Eng. Expt. Sta., Purdue Univ., 1920. **58.** *E. C.*, July, 1924, p. 71, 72. **59.** *J. A. W. W. A.*, Vol. 7, 1920, p. 541. **60.** *J. A. W. W. A.*, Vol. 10, 1923, p. 959. **61.** *E. N. R.*, Dec. 13, 1917, p. 1112. **62.** *Power*, Sept. 5, 1922, p. 364. **63.** *E. N. R.*, July 15, 1920, p. 113. **64.** *E. N.*, Sept. 7, 1916, p. 441. **65.** *J. N. E. W. W. A.*, Vol. 33, 1919, p. 157. **66.** *Can. Eng.*, May 22, 1923, p. 525. **67.** *Power*, Mar. 8, 1921, p. 376. **68.** *E. N. R.*, Nov. 4, 1920, p. 892. **69.** *Can. Eng.*, July 3, 1923, p. 102. **70.** *E. N. R.*, May 31, 1917, p. 446. **71.** *J. N. E. W. W. A.*, Vol. 33, June 8, 1919, p. 159. **72.** *Power*, Jan. 4, 1921, p. 17. **73.** *J. N. E. W. W. A.*, Vol. 37, 1923, p. 245. **74.** *Can. Eng.*, May 22, 1923, p. 525. **75.** *J. A. W. W. A.*, Vol. 8, 1921, p. 412. **76.** *E. N. R.*, July 25, 1920, p. 113. **77.** *E. C.*, Jan. 12, 1916, p. 46. **78.** *E. N. R.*, May 3, 1917, p. 256. **79.** Vosbury: *J. A. W. W. A.*, Vol. 12, 1924, p. 381. **80.** Burdick: *J. A. W. W. A.*, Vol. 11, 1924, p. 371. **81.** *Chem. & Met. Eng.*, Vol. 29, 1923, p. 928. **82.** Diesel Engine Users Assn., Apr. 7, 1922. **83.** *E. N.*, May 7, 1914, p. 1015. **84.** *E. C.*, Apr. 29, 1914, p. 498. **85.** *E. N. R.*, Apr. 3, 1919, p. 655. **86.** *Am. City*, 1924, p. 514. **87.** *Fire & Water Eng.*, Dec. 12, 1923, p. 1244. **88.** *J. A. W. W. A.*, Vol. 12, 1924, p. 391. **89.** Catalog, Rife Hydraulic Engine Mfg. Co., 1924. **90.** *E. N. R.*, Jan. 24, 1918, p. 174. **91.** *Power*, Oct. 9, 1923, p. 582. **92.** *T. A. S. C. E.*, Vol. 59, 1907, p. 257. **93.** *Reports*, 1912–1924, Chicago Public Works Dept. **94.** *Power*, Nov. 18, 1923, p. 986. **95.** Kreisinger, Mech. Eng., Vol. 43, 1922, p. 321–2. **96.** Savage, Gas Age, Vol. 75, 1920, p. 541.

CHAPTER 23

RESERVE STORAGE

IN DISTRIBUTION RESERVOIRS, STANDPIPES, AND TANKS

Function. Adequate reserve storage near the point of demand assures freedom from interruption of supply by failure of a pumping station, filtration plant, or long conduit. There should be at all times enough water stored both to furnish several fire streams and to meet the domestic peak demands for several hours or several days, depending upon circumstances. Reserve storage is intended primarily to equalize hourly and daily fluctuations; monthly and seasonal variations are generally met by regulation at source of supply. Other advantages* are: (*a*) reduced sizes of conduits, pumps, and filters; (*b*) higher pump efficiency, due to uniform head; (*c*) uniform draft from wells, with least disturbance of water-table equilibrium (see p. 243); (*d*) limitation of pumping to a few hours a day in small plants; (*e*) longer time for making repairs; (*f*) filling of reservoirs by small electrically operated plants during hours of low power demand at reduced rates (see p. 525); (*g*) improvement of insurance rating of the system; (*h*) greater public confidence in the system; (*i*) improvement of fire pressures by supplying the distribution system from more than one direction. Clear-water basins of treatment plants function as reserve storage, although often at a low elevation and of small capacity.

Location. The best location for reserve storage where fire requirements predominate is near the high-value district and on the opposite side of it from the source of supply, so that water can be delivered from both directions, with consequent gain in quantity and pressure. Feeding hydrants from two directions, instead of one, eliminates 75 per cent. of the loss of head. Location must also be governed by topographical considerations, to secure the required elevation of flow line, and by geological ones, to avoid excessive rock excavation. Generally sites free from rock are chosen, although the circular reservoir at Dubuque,[1] 41 ft. deep, goes 10 to 20 ft. into rock.

Elevation of flow line may be determined by fire-pressure requirements, by topography, or by pumping economics. The flow line must be high enough to give the requisite fire pressure with allowance for friction losses in the pipes under high flows and when the storage is drawn down several feet. Where there are two or more reserve storages on the system, flow must be regulated by check valves; see Needham experience in *E. N.*, June 1, 1916, p. 1030. Bear in mind that water is extracted at fire-demand rates so that friction losses are fixed by effluent rather than influent rates. Reserve storage can be filled at times of minimum friction losses. Wherever it is necessary to place the flow line below that elevation fixed by fire pressures, the reservoir is put out of service during a fire by closing valves on the reservoir feeder, commonly by electric control (see p. 443). Under these operating conditions,

* See also p. 469.

the reservoir provides a safeguard and equalizes domestic demands; but reliance for fire-fighting rests entirely on the auxiliary pumping equipment.

Essentials. Proximity to high-value district, at proper elevation; safe construction for populous districts; architectural and engineering design that will not depreciate real estate values; watertightness (leakage when visible leads to apprehension; leakage on concrete tanks results in unsightly spots; waste is costly because water has a higher value than in impounding reservoirs, since money has been expended on its conveyance; leakage may damage the surrounding high-value district); lining for reservoirs in earth or rock to prevent leakage, to facilitate cleaning, and, in covered reservoirs, to distribute roof loads; roof, under some conditions (see p. 538); control works (see p. 536).

Types Compared. Water may be stored in open or covered reservoirs, in standpipes, or in tanks on towers. Open reservoirs with earth embankments are cheapest for large quantities. A reinforced-concrete reservoir is likely to be tighter and safer than a lined earth embankment. Questionable watertightness of coarse gravel underlying a site commanding the high-value district at Duluth led to decision for a reinforced-concrete reservoir (see also p. 541). Standpipes or tanks on towers or buildings are necessary to store water at elevations required for fire pressures where topographical or other conditions prevent use of shallow reservoirs. They take less area, and may be near the high-value district, thus cutting down friction losses. Steel standpipes were much used until 1910, but not in recent years, due to lack of economy and to great danger of failure in high standpipes. Very tall standpipes of small diam. lack economy because the useful capacity is only that above the required elevation; water below serves only as support, but demands an expensive shell for its retention; a steel tower is cheaper and more suitable support.[83] The higher the storage elevation the more expensive the standpipe. An extremely high standpipe may cost several times a tank on tower. Standpipes are economical only where relatively large capacity is needed and the elevation of the ground permits the whole capacity to be useful. The disadvantage of dead storage in standpipes may be overcome in some situations by a dual tank, as at South Poland, Maine,[117] where an ingot-iron bottom is inserted near top to give a capacity of 26,000 gal. for sprinkler pressure with 400,000 below for ordinary supply. Ingot iron was chosen because of resistance to corrosion when alternately wet and dry. Elevated tanks in cold climates require frostproofing of riser pipes. They are ugly unless architecturally well treated. Their conical or hemospherical bottoms afford opportunity for sedimentation; blow-offs are provided for discharging the sediment.

Size Required. The standard schedule of the National Board of Fire Underwriters assumes a 10-hr. fire in cities over 2000 population and a 5-hr. fire for smaller towns. Most Boards of Underwriters recommend 350 per cent. of the maximum daily consumption.

Large City. In a large city reserve storage must be sufficient to equalize fluctuations in demand and supply and to restrict the operating of additional pumps.* The consumption rate throughout the 24 hr. usually varies less in large cities than in small communities, and a smaller percentage of equalizing

* See "Reservoirs vs. Additional Pumps for Peak Demands," by Quayle, *E. N. R.*, Aug. 25, 1921, p. 328.

capacity is needed. For 24-hr. periods, the excess consumption is not over 6 per cent. in Manhattan borough, and 9 per cent. in Brooklyn, New York City. The night flow is assumed at 50 per cent. the day flow; 12 per cent. from an equalizing reservoir would prevent undesirable fluctuations. An equalizing reservoir is much more necessary for a grade aqueduct than for a pressure pipe. New York City has 72 days' reserve in Hill View and Kensico reservoirs. The terminal reservoir of Los Angeles aqueduct holds 180 days' supply.

For a City of 50,000. Five methods of computing the requisite capacity: (1) John R. Freeman[4] recommends a storage capacity of 1,000,000 gal. for each 15,000 population, for 6 hr. to provide fire protection. (2) J. T. Fanning[5] reckons the combined fire and domestic consumption for a city of 50,000 as at the rate of 12,000,000 gal. per day. (3) Consider 18 fire streams, each discharging 250 gal. per min., operating for 6 hr. If the reservoir is to supply this emergency draft, it must give out $250 \times 60 \times 6 \times 18 = 1,600,000$ gal. Assume maximum domestic consumption at the same time. From pumping or other records (see p. 573) plot a curve of hourly variation of consumption; on this pick out the 6-hr. stretch giving the greatest consumption, and compare with the rate per day. It gives say 20 per cent. excess. If the average rate of consumption on a summer's day (or any other season of maximum use) is 8 mgd., the storage required in the reservoir is 0.25×120 per cent. $\times 8 + 1.6 = 4,000,000$ gal. (in 6 hr.). In the report of National Board of Fire Underwriters on a manufacturing city of 50,000 it is recommended that 8000 gal. per min. be available in the downtown district. For a 6-hr. fire, 2,800,000 gal. would be required. If the maximum domestic consumption were occurring at the same time, the reservoir supply must be $2.8 + 2.4 = 5.2$ million gal. in 6 hr.

Apulian aqueduct, Italy,[46] 152 mi. long, supplies communities located from 15 to 20 mi. from the aqueduct. Each community has a reserve storage amounting to 72-hr. supply if pumping is required, and 48-hr. for a gravity supply.

Tank or Standpipe. With a liberal allowance for increase in population, 30 gal. per inhabitant should be the minimum capacity; no tank for municipal

Table 162. Maximum Fire Streams Obtainable from Tanks at Various Elevations[7]

Indicated pressure at base of play pipe lb. per sq. in.	Effective fire stream		Gallons per minute	Height of tower required to maintain fire streams as shown in columns 2 and 3 through 2½-inch cotton rubber-lined hose of lengths as given below					
	Extreme height, feet	Extreme reach, feet		Hose lengths, in feet					
				50	100	200	300	400	500
25	44	44	188	72	80	100	119	137	156
30	52	50	206	86	96	121	142	165	186
35	59	54	222	100	112	140	165	190	218
40	65	59	238	116	128	161	188	218	......
45	70	63	252	130	144	180	204	......	......
50	75	66	266	144	160	201	......	......	......
55	80	69	279	158	176	......	......	......	......
60	83	72	291	172	192	......	......	......	......
65	86	75	303	188	208	......	......	......	......
70	88	77	314	202	......	......	......	......	......
75	90	79	325	216	......	......	......	......	......

supply should have a capacity less than 30,000 gal. For institutions, make a careful investigation of the water used and allow a large excess for fire protection. Capacity and height for fire protection to factory buildings are generally prescribed by the insurance companies. For industrial tanks, see *J. A. W. W. A.*, Vol. 6, 1919, p. 705.

Appurtenances.* Although reserve storage is not intended primarily to facilitate sedimentation, it often so functions, and provisions must be made for cleaning. A reservoir may have a lining, sloping to a sump, drains, and dividing walls so that parts of the basin can be cut out without interrupting service.

Pipes. The largest reservoirs have separate influents and effluents with means for drawing water at different elevations. Smaller reservoirs have but one pipe, the flow being reversed when demand exceeds supply. The inlet generally discharges at the far corner and the effluent is drawn near the gate house to prevent stagnation. The outlet should have a bell mouth to prevent entry losses and a strainer with large total area at openings.† Check valves should be provided to assure one-way flow.

Ladders. Standpipes and towers require ladders inside and outside for access. These should be heavily galvanized or of material that does not corrode or rot. Metal rungs should be 8 in. from wall, so as to come under instep with large boots; 6 in. not enough; rungs should be at least ⅝ in.

Sedimentation basins require special provisions for accelerating sedimentation and quickly removing the sediment. To secure uniform flow at low velocity and prevent stagnation, influent and effluent troughs are placed on top of opposite longitudinal walls, and controlled by weir boards; these are termed skimming weirs.

Baffles are commonly installed to divert currents downward or upward. They are generally of wood so that they may be readily modified as conditions dictate. Baffles are occasionally subjected, during emptying or filling, to several feet difference in head on opposite sides, and should be designed accordingly. Too rapid filling of the Toledo basin[8] built up a head of 5.5 ft. against a plain concrete baffle, causing it to collapse with parts of the roof.

For blowing off sediment, bottoms are commonly formed into hoppers; at the apex of each there is a quick-opening valve to the drainage system; by opening one at a time the effluent will wash out considerable sediment. Large basins must be put out of service and cleaned by men and trucks or machines. A Ford car hauling a plank drag brought sediment averaging 8 in. thick to the drain outlet at Trenton reservoir.[112]

Reservoir controls should include all necessary valves and gages. Where there is to be an electrical gage recorder (see p. 569), provide tubes for floats. There should always be overflows to drains or waste conduits. Each pipe line should be controlled by a valve. All valves should be operated from one floor, having extension stems, if necessary. Altitude control valves (see p. 451) are often installed to shut off the flow when the reservoir is filled. Valves are often electrically operated from the pumping station or other distant place, for cutting out the reservoir so that pressure may be increased for fires. Sioux Falls is typical; with the standpipe on the line, the pressure cannot exceed

* For water-level recorders, see p. 569.
† See p. 451.

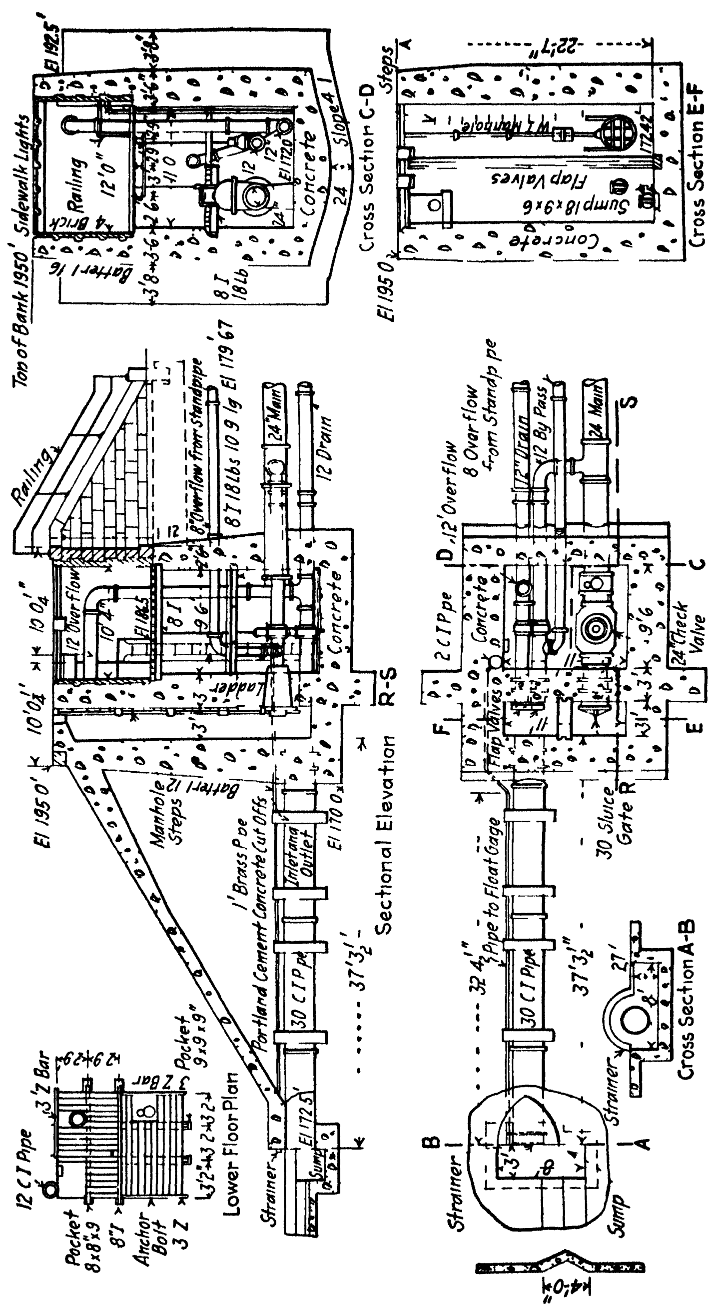


Fig 225 —Gate-chamber and pipe connections, Forbes Hill Reservoir, Quincy, Mass [10]

72-lb. To attain the 108-lb. fire pressure, the standpipe is shut off by hydraulic valves controlled electrically from the pumping station (for various forms of control chamber, see Angus, *Can. Eng.*, April, 1924, p. 14). Generally, the control works are in the embankment, or outside the reservoir, but occasionally as at Columbus, Ga.,[9] to secure best circulation of water, chambers are located in the reservoir and reached by boat, or bridge.

Covered vs. Open Reservoirs.* Modern practice favors covering of reservoirs if ground or filtered waters are stored. Otherwise the stored waters deteriorate due to growths of offensive microorganisms in sunlight, and frequent interruptions for cleaning materially affect the net income of the department.† Covering adds about 15 to 20 per cent. to cost.‡ On the ground of economy, Maury reported against roofing the Dubuque reservoir;[1] although conceding that the roof would prevent growths of algae, he claimed that these are harmless and can be cheaply prevented by other means. Benefits of covering (which apply as well to surface waters) are: reduction of evaporation, safeguarding from pollution,§ protection from freezing, maintenance of low temperature in summer, no modification of structure if water is eventually filtered. Roofs on water tanks, particularly small tanks on buildings, prevent them from becoming breeding places for mosquitoes, flies, etc. In cold climates roofs are commonly omitted from steel tanks on account of ice.

RESERVOIRS‖

General Classes. Geological conditions and structural preferences have resulted in types ranging from a dam across a valley to a covered reinforced-concrete structure above ground. The most common type is partly in cut, with embankments formed from the excavation, lined to minimize seepage and to facilitate cleaning. Where the material is unsuitable for embankments, the area limited, or much rock excavation required, it may be expedient to build a standpipe, a tank on a tower, or a reinforced-concrete reservoir.

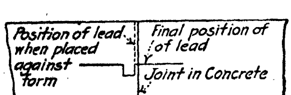


FIG. 226.—Queen Lane Reservoir, Philadelphia. Expansion joint in floor.

Depth of Water. Shallow reservoirs more readily affected by temperature variations, demand more frequent cleaning, and require longer walls and more real estate for a given capacity. On the other hand, shallow reservoirs reduce the rock excavation and unit wall costs, and give a more uniform pressure in the mains with less variation in the working load on the pumps. With earth embankments depth of reservoir and design of embankment should balance cut and fill. Depths of water are generally 12 to 18 ft. in reservoirs below 5-mg. capacity.

Lining.* A good watertight lining can be made of properly proportioned asphaltic concrete a few inches thick, similar to that used for highway surfacing, or of Portland cement concrete. Concrete may be monolithic, with

* See also p. 639.
† See Bunker and Nolte, *J. N. E. W. W. A.*, Vol. 37, 1923, p. 261.
‡ Maury[1] calculated in 1916 that a 7.5-mg. circular reservoir at Dubuque would cost $85,000 open and $102,000 covered.
§ "It has not been unusual to fish drowned cats, dogs, cows, and even babies out of uncovered reservoirs" (McDonnell).[3]
‖ Sedimentation and clear-water basins of treatment plants have the same structural features as reservoirs.

or without reinforcement, but thickness is best controlled from screed boards by laying in alternate squares on floors, and in strips on slopes. Top forms were used in constructing paving for channel at Sherman Island power house.[119] Floor blocks should be separated by three-ply tar paper; such floors can undergo some settlement without undue leakage. Lining thickness is dependent on geology. Professor Matthews[12] of University of London holds that 6 in. should be specified for water depths up to 15 ft., 9 in. from 15 to 25 ft., and 12 in. from 25 to 35 ft. Justin[13] says that blocks should be not larger than 6 by 6 ft., and thickness in in. should equal length in ft. Many linings have cracked under the weight of water. Slope paving is generally placed in alternate strips 5 to 8 ft. wide. Concrete can be cast fairly wet on a 1-on-2 slope.[14] Delay due to waiting for the lower concrete to set was avoided on the Omaha reservoir[15] by placing in horizontal strips around the rectangular basin; this allowed a wetter concrete than in vertical strips, of a consistence best suited to reinforced concrete without the slipping of the fresh concrete caused by pressure from the superposed material. A complete horizontal course was placed each day. Mix was 1:3:5, with integral waterproofing in the tempering water. Slope paving should abut against a curb, which should extend not less than 18 in. below the bottom of the paving.

Reinforcement of monolithic paving is commonly 0.3 per cent. of area of cross-section of concrete. Paving of reservoir at Portland, Ore.,[16] 5 to 6 in. thick, and reinforced 0.1 per cent. with $\frac{3}{8}$-in. rods 24 in. center to center, cracked before filling, and more cracks developed after filling. Reinforcement of 6-in. monolithic lining at Omaha[15] consisted of $\frac{1}{2}$-in. square twisted bars on 8-in. centers both ways, with lower bar $1\frac{1}{4}$ in. above bottom of slab. Bars were tied at every fourth intersection by special ties shaped to be used also as chairs. The monolithic lining for a small reservoir at Aberdeen, Wash.,[17] is 6 in. thick, consisting of 5-in. base with one bag of cement to 5 cu. ft. of concrete, and a 1-in. top of 1:2.5 mortar, reinforced with No. 28 triangular mesh. No noticeable leakage occurred at first, but in 1925 Supt. S. C. Watkins reported the leakage as "too great," and that "it is an absolute mistake to omit proper expansion joints." At Covington, Ky.,[18] the monolithic lining consisted of a 6-in. base mixed 1:2:4 and reinforced with wire cloth (No. 10 wires 4 in. center to center) 8 ft. wide, with longitudinal joints lapped 8 in.; a $\frac{1}{2}$-in. top coat of 1:2 mortar was applied after the base had set. Justin[13] questions the value of monolithic paving; settlement may allow slab to break, and leakage or wave wash cause failure of the embankment. When lining is cast in blocks,[13] reinforcement is commonly omitted. Slabs 17 ft. square and 6 in. thick laid with asphalt joints but without reinforcement were

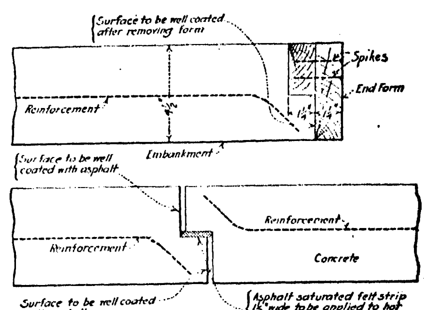


FIG. 227.—Expansion joint, floor of Vancouver Reservoir.

*See also J. A. W. W. A., Vol. 15, 1926, p. 118.

successfully moved about 18 ft. on rollers and replaced after subgrade repairs, without injury.[19]

Expansion joints are essential in linings exposed to great temperature variations, to relieve the stress; Seattle designs are shown in Fig. 229; the recent design was developed to prevent asphalt being forced between lining and rib by water pressure. Expansion joints my be replaced, but at loss of

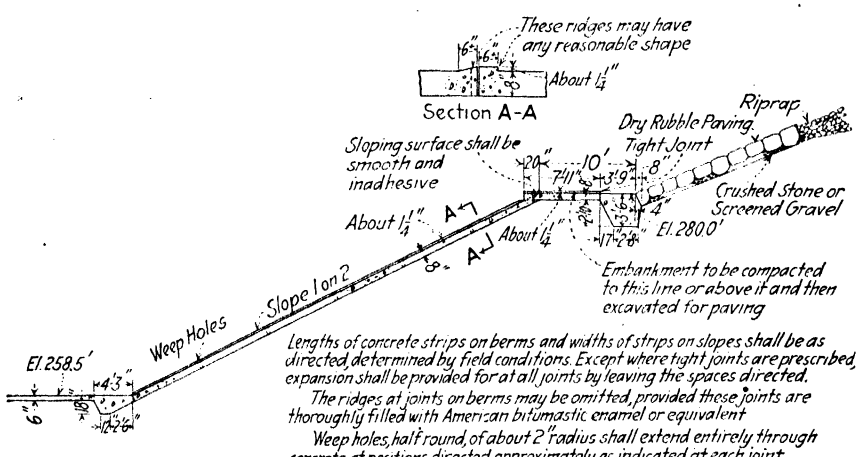
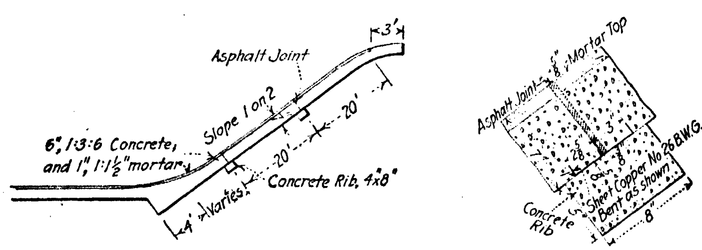


FIG. 228.—Hill View Reservoir, New York City. Typical details of linings. (Variations above berm are for landscape effect.)

economy, by adequate reinforcement. Authorities have never agreed as to the quantity of "temperature steel" required; considerable money can be involved in this item. See Table 163.

Failure of Lining. Many reservoirs with lined embankments are not watertight, leakage occurring (1) at intersections of slopes with bottom or berms; (2) at cracks in lining caused by excessive weight of water; (3) in the



A. Beacon Hill Reservoir,[26] 1911. *B.* Recent design of contraction joint.

FIG. 229.—Joints in lining at Seattle.

joints between slabs; or (4) through porous places in the lining. Puddled embankments and compacted floors add to watertightness.

Repairs to Lining. Leaks have been stopped by cutting V-shaped grooves 1½ in. deep, and of 1-in. maximum width, and filling carefully with ironite and cement;[1] by a superposed inner lining (Walnut Hill reservoir, Omaha,[27]

and Compton Hill reservoir, St. Louis[28]); or by coating of gunite (Kirkpatrick Hill reservoir, Nashville,[29] West Park pool, Pittsburgh,[31] Mt. Vernon, Iowa,[30] Herron Hill reservoir, Pittsburgh.[33]) Gunite has been used as the only lining at Muscatine, Iowa.[32]

Leakage under test from reservoir 180-ft. diam. and 40 ft. deep is reported by Maury as 32,000 gal. in 24 hr. All cracks were subsequently treated with a mixture of pulverized iron and cement, and structure under second test proved "as nearly watertight as it is possible to make such a structure."[1]

Construction plant is peculiar to each job; technical articles dealing therewith cannot properly be abstracted in a handbook. The plant on the Baldwin reservoir for Cleveland is described in *E. N. R.*, Nov. 30, 1922, p. 916. The planting for filtration plant construction and for covered reservoirs is somewhat similar, and much information can be acquired from articles on both subjects.

Reservoir walls are more costly than lined embankments but are justified where excavation must be saved, where there is rock, where tightness is essential, where the materials excavated are unsatisfactory for tight embankments, or where a roof must be supported.* Walls should be calculated to withstand all conditions of loading; a questionable practice, although utilized for groined-arch analysis, relies on the passive resistance of the earth backfill to reduce the effect of the water pressure.† Close analysis by Halmos[34] disclosed that the elastic deformation of the wall under full fluid and earth load would not produce sufficient passive resistance in the earth fill; the walls of the reservoirs at Three Rivers, Que., were consequently reinforced against water load. Equivalent fluid loads are generally used by designers in calculating earth-load stresses; these range from 30 to 35 lb. per cu. ft. The swimming pool at Camp Dodge, Iowa,[35] was calculated on the basis of 30 lb. and a weight of earth of 100 lb. per cu. ft. Dividing walls should be reinforced for either basin empty and the other full.

Gravity-type walls were used in many open reservoirs, but their lack of economy in resisting both conditions of loading (reservoir full and reservoir empty, with earth load acting) led to the use of reinforced-concrete walls. Reinforced-concrete construction‡ offers the designer many choices: (*a*) vertical slabs, (*b*) horizontal slabs, (*c*) vertical cantilevers, and (*d*) cylindrical tank. The most economical depth can be arrived at only by trial.

Vertical slabs are supported at the bottom by the floor, and at the top by a roof, or, in filter tanks, by a widening of the wall to form a beam, which serves as a part of the walks around the tanks. There is the advantage of simple form work on account of the uniform wall thickness, and straight bars. There is some economy in calculating as doubly reinforced, since reinforcement must be used in both faces to satisfy opposite conditions. Walls for oil reservoirs at Three Rivers, Que.,[34] were placed in grooves in the floor slabs, so that a slight angular displacement is possible and the walls could be treated as hinged top and bottom.

* Analysis of conditions at Enid, Okla.,[39] showed that a reservoir with slope paving surmounted by a low vertical wall is more economical than one with a full-depth vertical wall. Walls were designed as cantilevers with asphalt joint at roof to accommodate roof movements due to the large area.

† In designing Mankato circular reservoir, passive resistance was assumed only below a 6-ft. depth.[121]

‡ See 15-p. paper, including bibliography, by R. D. French, *J. Eng. Inst.*, Canada, September, 1919, p. 590; also Johnson on Cambridge filter tanks, *J. N. E. W. W. A.*, Vol. 37, 1923, p. 339.

Horizontal slabs are supported by counterforts. Counterforts add to cost of form work; reinforcing details are more complex; waterproof expansion joints between slab and counterfort are essential; counterforts on the inside interfere with the circulation of the water, and on the outside detract from appearance. Counterforts on the outside were masked at Muskogee[36] by a curtain wall. To secure the stabilizing effect of the water load on the counterforts, slabs at Muskogee,[36] Austin, Tex.,[37] and Highland Park, Mich.,[38] were inclined to the vertical and rested on the inclined face of the counterfort; a close analysis of foundation pressures is necessary. The buttressed circular type was estimated the most economical for a subsurface reservoir at Dubuque;[1] the counterforts rest on rock.

L-shaped walls, vertical cantilevers, have been the subject of patent litigation* but have the advantage of simple form work and utilizing the stabilizing effect of the load retained; there is some saving in reinforcing steel.

Cylindrical tanks, circular reservoirs, are easy to calculate, economical to reinforce, require least wall length for a given enclosure, and eliminate the weakness at corners inherent in rectangular reservoirs. Milinowski[40] cites as particular advantages of circular reservoirs below ground: Light reinforcement possible† removes the internal shrinkage stresses tending to cause cracks in the concrete; the solid earth backing puts the concrete in initial compression; the masonry is protected from drying out‡ and from great temperature ranges, so that stresses from these causes are reduced; leakage from underground reservoirs is resisted by the groundwater head, and produces no exterior indications as in tanks above ground to alarm residents; such tanks are more readily waterproofed than rectangular tanks. Disadvantages of circular tanks are: with beam-and-slab roof, form work and constant variation in lengths of reinforcement next to the wall, as exemplified in the Hibbing, Minn.[41] tank, add to the cost; low value assigned to the reinforcing steel, to prevent cracking the concrete; cost of forms (met at Enid, Okla.,[39] by using an octagonal plan to give easier form work); difficulty of architectural treatment, particularly on a plain concrete surface (see also p. 556).

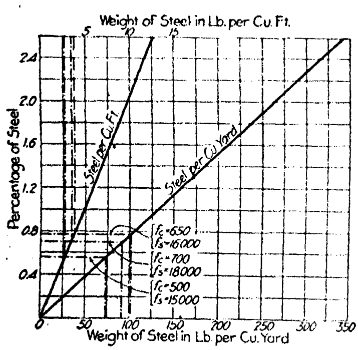


FIG. 230.—Diagram for estimating weight of steel in concrete.[43]*

* Add to this, temperature steel: *e.g.*, ½" round, 12" c. to c. in each face of a 12-in. wall amount to 0.27 per cent.

Reinforced-concrete designing for reservoirs requires the same faithful attention to details as in other reinforced-concrete construction, with the added necessity of producing a watertight structure. Pipe openings should be investigated and extra reinforcing put in. Twisted square bars were excluded from St. Paul reservoir,[42] and deformed bars only specified, on the basis of Talbot's findings in University of Illinois tests.§ The thickness of low walls

* See E. N., Mar. 2, 1916, p. 429.
† The Dubuque reservoir,[1] 10 to 20 ft. in rock, was not reinforced in rock.[1]
‡ See p. 794.
§ See *Bull.* No. 8, Eng. Expt. Sta., 1908.

is commonly made greater than called for by analysis, to give a greater working space for placing the reinforcement and the concrete. The steel content of the walls, therefore, varies within rather wide limits. The steel content may be estimated from Fig. 230. A common figure for Imhoff tanks, including temperature steel, is 100 lb. per cu. yd. The swimming pool at Fort Dodge, Iowa,[35] averaged but 67 lb. reinforcement per cu. yd. The question of the amount of shrinkage (or temperature) steel is economically important, as money can be wasted on it. Shrinkage steel data taken from plans of various filter tanks (the walls are designed as vertical slabs) are given in Table 163.

Table 163. Shrinkage Reinforcement in Filter Tanks

Plant	Rod size, in.	Spacing, in.	Side wall: Thickness, in.	Side wall: Per cent ratio to area	End walls: Thickness, in.	End walls: Ratio to area	
Wilmington	3/8	18	9	0.0013	12	0.00097	Actually used 1/2" ϕ 18" c. to c.
Minneapolis	1/2	24	9	0.00116	12	0.00087	
Jerome Park	1	18 side walls; 12 end walls	9	0.00154	12	0.00174	
Cleveland	1/2	12	12	0.00174			
Baltimore	1/2	12	9	0.00231	12	0.00174	
Columbus	1/4	9	9	0.00077	18	0.00038	
Eastview	1/2	18	9	0.00154	12	0.00116	

Floors of reinforced-concrete reservoirs differ from those for lined embankments only when roof columns have to be supported and the load distributed to the foundation. Where the ground-water level is high, the floor must be designed to resist uplift when the reservoir is empty, by either groined arches or reinforced-concrete slabs, carrying the load to the columns. Commonly, groined-arch roofs and floors go together, although the recent 10-mg. reinforced-concrete reservoir for Indianapolis has flat-slab roof and groined-arch floor.[44] Groined-arch floors can be more readily formed into valleys for quick draining. Column loads are often distributed by reinforced-concrete grillages, the floor being deepened at each column footing. Form a waterproofed joint around column footing.

Foundations are important. Settlement of the underlying earth under heavy loads destroyed part of the new filter house at Cleveland.[45] A heavy pumping station adjacent on a substantial pile foundation showed no settlement. The earth was a miscellaneous fill, which had been tested by loading platforms in 1914 and judged to be good for 1.5 tons per sq. ft. Reservoir floors, particularly in filter plants, often rest on roofs of clear-water basins, giving large foundation pressures. Subgrades are often consolidated with 5-ton rollers. Placing concrete on wet, earthy material weakens the concrete. On wet subgrade it is good practice to lay a mat of lean concrete, of a thickness determined by conditions, before placing the floor. At St. Paul,[42] a mat half

clay and half sand and gravel, mixed, rolled in 6-in. layers, was placed over very pervious sand and gravel subgrade. Although the reservoir at Greenfield, Mass.,[48] was founded entirely on trap rock, bottom leaked badly until grouted, due to shrinkage of floor slabs and wall sections, which were poured separately. Joints were calked with jute packing to retain the grout. Oil reservoirs in southern California for Empire Pipe Co.[47] were excavated 1 ft. below grade and backfilled with selected clay to furnish a uniform backing for the concrete lining, which was laid on a 2-in. sand cushion to take up settlement or swelling.

Roofs are commonly concrete groined arches or reinforced-concrete slabs or wood. Roofing detracts from the economy of circular reservoirs but makes for economy in rectangular reservoirs and has led to the completely reinforced concrete reservoir. Earth cover may be required to increase frost protection, or for landscaping effects. Tennis courts are laid out on roof of reservoir at Arkansas City, Kan.[120] A 24-in. earth cover at St. Paul was not sufficient to prevent $\frac{3}{4}$-in. ice occasionally.[42] At least 20 in. is required to support vegetation. Wooden roofs may be used where there are no snow or earth loads. Such heavy loads require masonry roofs; the necessity for taking the wall reaction has also required masonry roofs.

Wooden roofs have short life unless the wood be treated; heavier walls are required as a wooden roof cannot take top reaction of thin walls. In reservoirs with sloping sides there are long spans over the slopes, or columns have to be placed on the slopes, a questionable practice. At Riverside, Cal., columns, 10 × 10 in., of reinforced concrete, at corners of 15.4-ft. squares, support 4- × 12-in. girders, on which rest 2- × 10-in. joists on 30-in. centers, supporting 1-in. boards covered with Johns-Manville standard roofing.[49] Unstable bearing on tops of slopes caused failure of wooden roof of Pasadena reservoir.[50] The timber roof at Quincy, Ill.,[51] consists of 6- × 6-in. girders of 18-ft. span supported by 6- × 6-in. columns, 22 ft. high. Joists: 2 × 8 in. on 4-ft. centers and 14-ft. span, with 2- × 4-in. bridging and 1-in. roofing. Reinforced-concrete posts, 8 × 8 in. up to 25 ft. unsupported height, carry 6- × 12-in. wooden girders in reservoirs of East Bay Water Co., Cal.[52] Floating roofs of wood have not been generally successful. Oil reservoirs for Empire Pipe Co.[47] in southern California have roofs of $2\frac{1}{2}$-in. concrete reinforced with metal lath, supported on 2- × 12-in. wooden rafters spaced 20-in. centers. Rafters are radial and rest on 10- × 18-in. girders, supported by 10- × 10-in. wooden posts. The wooden roof of Santa Monica reservoir[53] is supported on 8- × 8-in. redwood posts on 16-ft. centers.

Groined Arches. Groined-arch construction for the roof and floors relies for stability upon arch action, but reinforcing steel is sometimes used to prevent cracking. Each pier supports four segments of barrel arches, at both top and botton. In the end bays, the arch reactions are taken by a continuous semiarch, so that one-half the end bay has a cross-section similar to half an aqueduct. The center lines of the bays are elements in the intradoses of abutting arches, said element forming a straight horizontal line. The controlling stresses are (1) punching shear in roof at the pier cap; (2) compression in the arch at crown; (3) compression at base of column. Sidewalls for St. Paul reservoir,[61] which has groined-arch floor and roof, consist of inclined

slabs supported by buttresses on three sides, and on the fourth side, which will eventually become a dividing wall for a future basin, there is a cantilever gravity wall. The elliptical intrados best meets the pressure-line requirements. The parabolic line of the intrados is rendered less steep at the haunches by substituting a 1-on-4 slope; in construction, it was found difficult to form concrete to the steeper slope; it settled into the haunch and caused an undesirable thinness in the arch.

In designing the half-arch end walls, practice countenances the assumption that the backfilled earth can sustain a passive pressure of 70 lb. per sq. ft.

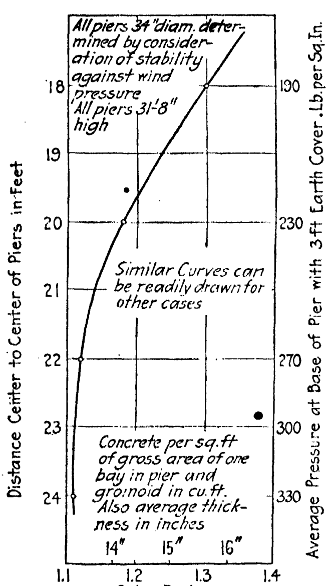


FIG. 231.—Economic span for groined arches.

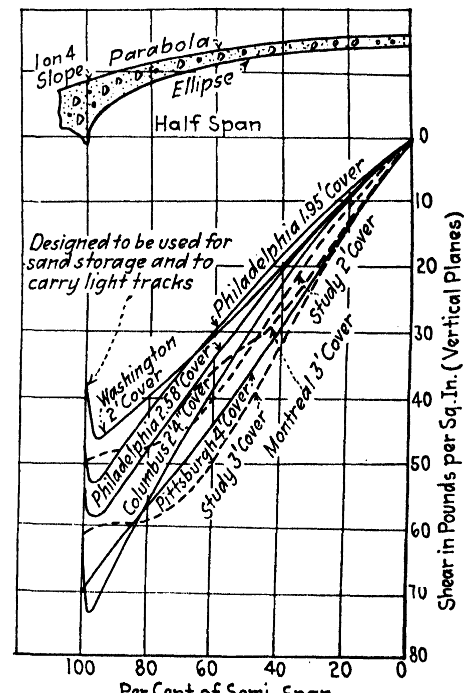


FIG. 232.—Punching shear* in existing groined arches (100 lb. per sq. ft. live load).

* Punching shear is similar to that in steel plates when resisting rivet-hole punch.

This requires the backfill being placed before the earth cover is superposed on the roof. I-beam groins encased in concrete are used in the new Cross Hill reservoir, Birkenhead, England,[54] to produce "the most economical reservoir in the world." The end walls are formed by a series of vertical brick arches transmitting the load to an earth embankment.

Method of Comparison. Assuming that this type of structure stands up through arch action, and not cantilever, and that joints are formed along the diagonals, each pier would be the abutment for four arches, the halves of which would be trapezoids in plan. By drawing a line of resistance and computing stresses in concrete, this method may be used as a comparison, although it is understood that the stresses are not necessarily those which obtain.

Table 164. Data on Concrete Groined Arches for Reservoir Roofs*[55b]

No.	Date	Location	Clear span (2a), in feet.	Rise of intrados (b), in feet	Crown thickness (t), in feet	Rise of extrados (h), (h), in feet	Span, center to center, of columns (2c), in feet	Ratios: $\left(\frac{b}{2a}\right)$	Ratios: $\left(\frac{h}{b+t}\right)$	Average thickness of roof. in feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	1903	Watertown, N. Y.	10.000	1.500	0.500	0.833	11.500	0.150	0.417	0.562
2	1909	Providence, R. I.	10.250	2.500	0.500	1.250	11.917	0.244	0.417	0.635
3	1903	Ithaca, N. Y.	10.500	1.500	0.500	0.833	12.000	0.143	0.417	0.568
4	1909	Springfield, Mass.	11.333	2.000	0.500	1.250	13.000	0.177	0.500	0.566
5	1911	Toronto, Ont., Canada	11.333	2.000	0.500	1.167	13.000	0.177	0.467	0.582
6	1910	Owen Sound, Ont., Canada	11.500	2.500	0.500	1.167	13.000	0.218	0.388	0.625
7	1903	Washington, D. C.	11.833	2.500	1.500	1.417	13.667	0.211	0.472	0.606
8	1899	Albany, N. Y.	11.917	2.500	0.500	0.500	13.667	0.210	0.167	0.774
9	1900	Superior, Wis.	12.000	2.500	0.500	0.500	13.667	0.208	0.167	0.747
10	1903	Washington, D. C.	12.167	2.500	0.500	1.417	14.000	0.206	0.472	0.600
11	1912	Roland Park, Md.	13.000	2.667	0.500	1.500	14.667	0.205	0.473	0.593
12	1907	Philadelphia, Pa.	13.167	3.000	0.500	1.750	15.000	0.228	0.500	0.609
13	1907	Lawrence, Mass.	13.167	2.750	0.500	1.667	15.000	0.209	0.513	0.590
14	1908	Pittsburgh, Pa.	13.167	3.000	0.500	1.750	15.000	0.228	0.500	0.609
15	1904	Philadelphia, Pa.	13.417	3.000	0.500	1.750	15.250	0.224	0.500	0.607
16	1913	Baltimore, Md.	13.500	3.000	0.500	1.750	15.000	0.222	0.500	0.582
17	1912	Grand Rapids, Mich.	13.833	3.000	0.500	1.833	15.000	0.217	0.524	0.543
18	1907	Philadelphia, Pa.	14.000	3.000	0.500	1.750	15.833	0.214	0.500	0.604
19	1902	Milford, Mass.	14.000	3.000	0.500	1.500	16.000	0.214	0.428	0.658
20	1909	Springfield, Mass.	14.000	2.750	0.500	1.500	16.000	0.196	0.462	0.623
21	1916	Cleveland, Ohio	14.080	3.500	0.500	2.000	15.750	0.249	0.500	0.618
22	1908	Columbus, Ohio	15.167	3.167	0.500	1.917	16.833	0.209	0.522	0.568
23	1905	Washington, D. C.	15.500	3.500	0.500	2.083	18.000	0.226	0.522	0.646
24	1908	Pittsburgh, Pa.	15.750	4.000	0.500	2.167	18.000	0.254	0.482	0.683
25	1912	New York City	16.000	3.750	0.500	2.000	18.000	0.234	0.470	0.653
26	1913	Minneapolis, Minn.	16.333	3.500	0.500	2.000	18.000	0.214	0.500	0.600
27	1913	Montreal, Que., Canada	17.000	4.250	0.500	2.000	19.000	0.250	0.422	0.700
28	1912	New York City†	18.000	4.500	0.500	2.500	20.000	0.250	0.500	0.646
29	1912	New York City†	19.160	5.000	0.500	2.580	22.000	0.261	0.469	0.744
30	1920	Cleveland, Ohio†	17.960	4.500	0.500	2.500	20.292	0.253	0.500	0.675
31	1923	Cleveland, Ohio, Baldwin‡	17.792	4.600	0.500	2.500	20.292	0.258	0.490	0.500

* See also table by Wiggin, *Proc. Nat. Cement Users Assn.*, 1910, p. 246.
† Under construction or proposed.
‡ Unusual height of 40-ft. floor to roof.

Table 165. Comparison of Stresses in Groined Concrete Arches*

Date	Location	Depression over pier	Pier section	Volume† saved by depression: Cu. yds.	Volume† saved by depression: Average depth	Mean thickness of roof	Max. pressure at pier base, lbs. per sq. in. incl. 100 lb. per sq. ft., live load	Height of piers	Arch stresses, lbs. per sq. in. (compression): Crown	Arch stresses, lbs. per sq. in. (compression): Joint of rupture
		in.	in.		in.	in.		ft. in.		
1903	Philadelphia, Torresdale	21	22 sq.	2.70	3.5	7.2	265	14–4	122	259
1905	Columbus	23	20 sq.	3.36	3.85	7.0	330	11–8	134	262
1903	Washington, D. C.	24⅜	30 sq.	4.0	4.0	7.9	180‡	25–7	115	234
1904	Pittsburgh	26	27 d.	4.33	4.33	8.25	380	21–6	163	267
Proposed	Jerome Park, New York	30	24 d.	6.2	5.0	7.75	410	23–0	— — —	— — —
Proposed	Study, B. W. S. New York	31	34 d.	9.25	6.2	8.3	330	31–8	165§ 131‖	294 233

* See also *E. R.*, Nov. 15, 1913, p. 539.
† For method of estimating volumes, see Paaswell, *E. N. R.*, Dec. 14, 1922, p. 1024.
‡ Roof designed to be used for sand storage. § 3-ft. load. ‖ 2-ft. load.

Tables 164 and 165 show the results obtained for various groined arches; 100 lb. per sq. ft. live load are assumed in all cases. (For detailed methods of computing, see *J. N. E. W. W. A.*, Vol. 14, 1899; "The Groined Arch in Filter and Covered Reservoir Construction," T. H. Wiggin (National Assn. of Cement Users), *E. N.*, Apr. 7, 1910; Macqueen, *T. A. S. C. E.*, Vol. 86, 1923, p. 182.)

Concrete Proportions. A mix of 1:3:5 was used at Philadelphia and Pittsburgh, and proposed for Eastview filters. At Columbus, Gregory used 1:2.5:5.5; at Washington, 1:2.9:5 was used.

Failure. Many groined arches are damaged by contraction and settlement cracks, causing leakage; recent practice favors substantial reinforcement.[56]

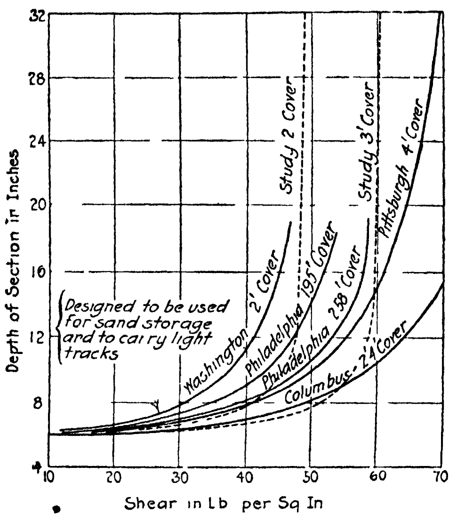


FIG 233.—Variation in intensity of vertical shear with thickness of section in existing groined arches. (Earth cover and 100 lb per square foot live load.) (See page 546.)

Ely[57] reasoning from failure of Belmont filters, Philadelphia, considers groined arches as neither safe nor stable unless tied together with steel. The groined-arch roof at Cleveland filter plant[58] failed after reservoir had been filled but before roof had received the 3-ft. earth load, evidently due to lax construction methods, as the concrete "appeared to be innocent of cement to an astonishing degree."

Groined Arch vs. Reinforced-concrete Roof. Whether to use a groined arch or a flat slab roof for a reservoir frequently involves differences of opinion. Both are acceptable, although, in certain cases, one is preferable. For small, shallow reservoirs of, say, less than 1 acre, the flat slab roof may be more economical, on account of the lower cost of form work, and, perhaps, better standardized methods of construction. For larger and deeper reservoirs, covering several acres, the groined arch will prove much more economical than the flat slab" (Macqueen).[55c]

A flat-slab roof was selected for Indianapolis,[44] as local builders were more familiar with this type and lower prices were expected. Roofs of fuel-oil reservoirs, Three Rivers, Que.,[34] are of two-way drop-panel flat-slab type, 16-ft. span, supported on square columns. Savings on Dayton reservoir[59]

from flat slab were credited to less excavation and simpler form work; the economies were questioned by some authorities.[56] The groined arch also involved 12 per cent. more concrete. Atwood[60] finds that the relative economy is decided by the loads for which designed; where the load exceeds 2 ft. of earth, there is economy in the groined arch; with less fill, slabs prove cheaper. A groined arch must be loaded with some care to prevent failure of the structure; a flat slab is free from this danger. Drainage of the depressions over columns is another difficulty in groined arches. Burdick[56] considers a completely reinforced reservoir a much better and more dependable structure. Non-reinforced work commonly cracks and leaks. Freedom from air pockets is an advantage of both flat-slab and groined-arch roofs. Slab-and-girder roofs are preferred by McClintock[56] to the girderless type due to the possibility of more accurate analysis of stresses, and greater safety around openings in the roof.

Columns. Usual spacing ranges from 8 to 25 ft., thickness of flat slabs from 5½ to 12 in. Columns at Indianapolis[44] were 24-in. diam. reinforced with eight ⅝-in. round and ¼-in. tie rods. The roof was figured for total superposed load of 350 lb. per sq. ft.

REINFORCED-CONCRETE STANDPIPES*

Steel vs. Reinforced Concrete. Reinforced-concrete standpipes were proposed to escape the maintenance for steel tanks, and to present a more pleasing appearance. Expectations were seldom realized. Concrete has not, in many instances, proved durable, watertight, or sightly. Johnson[62] believes that "concrete standpipes are far from indestructible." Some engineers despair of making a concrete tank tight.[40] Some have not leaked; Andrews[63] reported no trouble from seepage when he limited the height to 60 ft. and increased the mix to 1:1:2. Woonsocket tank, built in 1912, was reported in 1925, as satisfactory; the city is contemplating another tank. West Falmouth reported in similar terms. Milinowski[40] says that there have been "hardly any" absolute failures by major structural breaks. Reinforced-concrete standpipes over 50 ft. high in localities having much freezing weather have not proved satisfactory.[62] Maintenance cost is likely to be greater on steel, although leakage is more easily stopped. Reinforced concrete can be more quickly erected; but there will be greater delay for plans, as manufacturers have available stock drawings of steel. Architectural possibilities are less with steel. First cost of reinforced concrete is generally greater, although[66] nearly the same. Sampson[82] made the following comparison for Middleborough in 1915: For a storage of 500,000 gal. with flow line 165 ft. above ground: (*a*) Steel standpipe 40 ft. in diam. and 160 ft. high would have a total capacity of 1,500,000 gal., a first cost of $44,000, and a high maintenance cost, besides inconveniences of direct pumping into the mains every time the standpipe was out of service for painting. (*b*) Concrete standpipe of same dimensions would cost $39,000; it might be impracticable to get it tight under 160-ft. head. (*c*) A steel tank of 500,000-gal. capacity on columns would cost $22,000, and would have a high maintenance charge besides inconvenience

* Based partly on paper by Andrews, *J. A. E. Soc.*, June, 1911, and papers in *J. N. E. W. W. A.*, Vol. 29, 1915, pp. 169–201, and *E. N.*, Mar. 18, 1915, p. 554. For merits of standpipes, see pp. 193, 199.

of direct pumping when out of service. (*d*) Concrete tank on a cylindrical tower was estimated at $24,000 with no cost or inconvenience for cleaning and painting; also it could preserve a more equable temperature in the water.

Dimensions. Tank dimensions should be carefully considered. Mensch[64] claims that a tank 40 ft. in diam. and 60 ft. high is considerably more expensive than a tank 50 ft. in diam. and 38 ft. high; capacities are same, but greater pressures may be obtained from the more costly tank. Height is limited by difficulty of getting tight work under high heads without a membrane. Simpson[62] is ready to build up to 115 ft. high, but not without an interior membrane. There was built in Kansas City in 1920 a tank 133.5 ft. high, 40 ft. in diam., and with maximum water depth of 110 ft. Capacity 1,000,000 gal. Leakage required the subsequent placing of a membrane.[65] Johnson[62] has never built tanks over 40 ft. high, and "all leak more or less." In 1913 there was built at Brunswick, Maine, a tank 97 ft. in diam. and 46 ft. high, holding 2,500,000 gal. The Lexington standpipe is 104 ft. high.

Calculations. Early standpipes were calculated for steel stresses by "thin-cylinder" formula at 12,000 to 16,000 lb. per sq. in. Manchester standpipe is designed on 12,000 basis. Deformation of steel stressed in excess of 4500 lb. per sq. in. exceeded the ultimate elongation of plain concrete and necessitated concrete cracking. This cracking is to be avoided as it leads both to leakage and to corrosion of the reinforcing steel. A first principle of this class of construction now is so to proportion all parts that the tensile strength of the concrete shall not be exceeded. Contraction in setting places the concrete in initial tension and the steel in initial compression. This consideration has led, within the past 10 years, to very rich concretes, such as $1:1\frac{1}{2}:3$, $1:1:2$, working stresses of 3000 to 6000 in the steel, and tensile stresses allowed on concrete of 300 to 500 lb. per sq. in. Messrs. Weston and Sampson believe Andrews' theory* results in a prohibitive cost for large standpipes. In the tank for Rockland, Mass., especially rich concrete was used; thickness of wall which would insure that ultimate strength of concrete would not be reached with full reservoir; increased vertical reinforcement, particularly at base. Reinforcement in the Attleboro tank was proportioned for an ultimate stress of 13,500 lb. per sq. in. Not to have the bars too close together, a double ring of $1\frac{1}{2}$-in. bars was used in first 61 ft., a single row of $1\frac{1}{2}$-in. bars for next 20 ft., and a single row of $1\frac{1}{8}$-in. bars for upper 19 ft. Spacing in the top 15 ft. was kept constant to take possible ice loads. In the Westerly standpipe, stress in lowest foot was 6000 lb. per sq. in., increasing 1000 lb. for each foot in height until maximum of 12,500 was reached. Wason found that steel more than 2 in. from a surface will not prevent cracking the surface. Hewett[67] favors keeping shrinkage stresses to a minimum and limiting the tension in the concrete. At Barnum, Minn., he first constructed an interior shell of concrete $7\frac{1}{2}$ in. thick and, after this had set, placed circumferential reinforcement bars equipped with turnbuckles for putting them in initial tension of about 14,600 lb. per sq. in. The reinforcing was later protected by being encased in 3 in. of concrete held in place by wire netting fastened to the bars. The interior of the tank was treated with two brush

* See J. A. E. S., June, 1911. The allowable stresses specified in the preceding sentence are based on Andrew's theory.

coats of "Ironite." "Not the least dampness ever appeared on the outer surface." Hooping has been utilized on oil-tank design (see *E. N. R.*, Sept. 11, 1924, p. 422). On Duluth reservoir Turneaure[2] assumed both concrete and steel to take tension in ratio of 1:10. Wall was thickened towards bottom to restrict tension in concrete to 240 lb. per sq. in. As a precaution against ice, lower steel stresses were used near the water line.

Thickness of wall is governed by considerations of watertightness, eveloping the steel, and allowing room for placing the steel and its proper enveloping in concrete. The minimum at top should be 6 to 12 in., depending on diam. For Manchester, Mass., standpipe, the wall is 20 in. at base and 12 in. at top. Wall thickness in the 100-ft. Attleboro tank tapered from 18 in. at bottom to 8 in. at top.

Proportions. Concrete has been increased in richness to add to tensile strength and prevent cracks, as well as to improve density. The few small leaks in the Attleboro tank indicated to the early designers that a 1:2:4 mix properly placed can be made watertight, although it may be better judgment to use a richer mix.

At Westerly, R. I., concrete was mixed to make only 12 cu. ft. to a four-bag batch, of 20 parts by weight of cement, 40 parts of No. 4½ sand, and 40 parts of No. 3 stone, screened to remove everything below ½ in. This mixture had the maximum density of all measured and appeared satisfactory. The curve given by this mixture, however, on volumetric tests, was nearly 10 per cent. higher than the "Ideal" at the point of tangency. With the densest mix, the addition of hydrated lime increases the volume almost directly in proportion to the lime used, and therefore is of little use.

Proportions recommended by Mensch:[64] 1 cement to 1.3 aggregate for heads of 80 to 100 ft.; 1:1.5 for heads 50 to 70; 1:2 for heads 30 to 40; and for less heads, 1:2.25 mix. These represent practice in France where concrete tanks have been in use 50 years, and where thin walls are the rule. Even rich mixtures and careful workmanship will not prevent leaks in construction joints.* An absolutely watertight tank can be assured only by skilfully applied waterproofing. Mix used at Merrimac, N. H.,[68] was 1:1.5:3 local sand and gravel, sizes ¼ to 2½ in. Gravel screened and washed. Specified mix for oil tanks in Canal Zone[69] was 1:2.25:3.75 (based on analyses of materials available), but had to be amended to 1:2:4 when the latter was proved to lack flowability to fill all spaces around the complicated reinforcing. A 1:2:4 mix at Waltham with especially selected aggregate (built in 3-ft. lifts) did not prevent joint leakage. From experience gained at Waltham it was decided to use at Manchester a rich mix, 1:1.5:3, and to increase reinforcement at base. Five per cent. hydrated lime was added at Waltham and Manchester. 1:1.5:3 was used at Kansas City;[65] limestone aggregate, ¼ to 2 in. The leakage (110-ft. head) required subsequent placing of membrane.

Reinforcing Steel.—Rings have been of plain or deformed rods, tees, or other shape; ends lapped, fastened with clips, clamps, or similar devices, or welded; one or two rings in width of wall, according to needs and judgment of designer. Spacing of rings and size of metal in them have been varied in the vertical according to decrease in stress, from bottom to top, but some mini-

* See p. 554.

mum size of metal and maximum spacing have been used for some set distance below top.

On the Attleboro tank, bars were obtained long enough so that three would reach entirely around, with a lap of 40 diam. at each joint. Two wire rope clips were used at each splice to insure bars being held firmly together. By the test at the Watertown Arsenal, these clips alone were sufficient to insure full working stress of the bare bars; any additional bonding stress from the concrete was an added factor of safety. Splices were staggered so that the eleventh laps came in the same vertical plane. Vertical steel of a variety of shapes has been used to support rings and resist moments, shears, and other stresses. On the Attleboro standpipe, 4-in. channels with $\frac{3}{8}$-in. holes through both flanges were set upright at intervals of 15 ft. with the web radial; a $\frac{1}{4}$-in. rod passed through both holes supported on its outer end in a hook, the hoop steel. No vertical steel was used on the Westerly tank. Ambursen[70] found in repairing a standpipe that there were air pockets underneath the larger bars where they had not been properly surrounded with concrete. Water seeping into such pockets leads gradually to disintegration. His experience leads him to advocate that "square bars over 1 in. thick should not be used in walls less than 12 in. thick." Advantage should be taken of the advance made in electrical and gas welding to weld tension-reinforcing steel and metal water stops so as to avoid awkward and uncertain laps, clamps, and other devices. Incidentally the saving in metal, labor, and troubles would go a long way toward paying for welding and might show a total net economy.

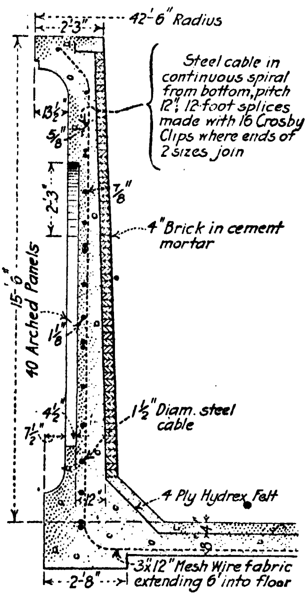


FIG. 234.—Cos Cob Reservoir. Wall section.

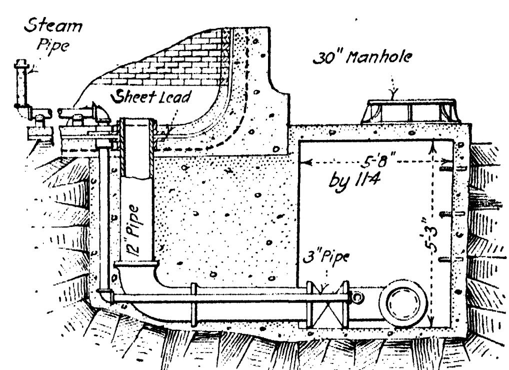


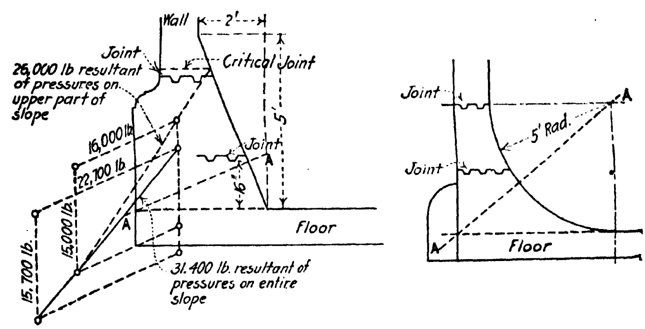
FIG. 235.—Cos Cob Reservoir. Section of valve chamber.

(Founded on rock. Capacity, 600,000 gals. N. Y., N. H. & H. R. R.)[122]

Construction. Standpipes have been built without forms. One method developed in Oregon[71] consists of plastering 1:2.5 mortar on triangular mesh. Ten per cent. hydrated lime was added to mortar, which was applied in thin coats. A 17,000-gal. reservoir is reported as "quite watertight" after 1 year. Cement gun has also been used in connection with heavy duck to limit the

thickness.[72] Leakage troubles have generally developed in joints between days' work; such joints were very carefully finished on the Westerly standpipe; two men were kept on into the night after concrete was placed to scrub the top surface carefully with brushes and water to get the laitance off and wash the surface of all stone showing on top as clean as before being mixed, because cement bonds well to clean aggregate, but does not bond well to old cement. Roughing strips were put in the wall to form grooves. These were also removed in the evening. Before concrete was placed the following morning, the surface was thoroughly washed with water, then coated with neat cement, rubbed in with a brush, and grooves were filled with neat cement.

To do away with joints, the 50,000-gal. standpipe at Merrimac, N. H.,[68] was poured in one continuous operation covering 39.5 hr. At no stage did more than 1 hr. elapse between successive layers. Tank was but 40 ft. high, and speed was attained by erecting complete the outside forms, thoroughly



Westerly, R. I. Attleboro, Mass.

FIG. 236.—Floor-and-wall joint.

braced to eight telephone poles erected around the circumference. This did away with any bolts, wires, or ties in the concrete.

Slipping the forms continuously up the standpipe, as was done at Fulton, N. Y.,[74] and keeping the forms full of fresh concrete, obviates construction joints between days' work. At Fulton, the forms moved about 5 ft. per day. Milinowski[40] complains that these slip forms are known to rupture concrete and start leaks. The 110-ft. shaft at Kansas City[65] was built in one continuous concreting operation.

The Attleboro tank was kept filled with water during construction to a level about 20 ft. below where concrete was being placed, principally as a protection to workman. Water was put into Bridgewater tank 10 weeks after foundation was started; the most recent concrete was then about 1 month old. Prison labor, not skilled in concrete, was used.

Floor-and-wall Joint. As there is a tendency for the diam. of a reservoir to increase after it is filled with water, due to the elasticity of the steel in tension, and as the base of the reservoir is practically rigid, due to contact with foundation, some reinforcing material extending from the base up into the walls to take care of the bending moment and shear at the base has been necessary. There is no exact method of obtaining the amount of steel required here, but it has been underestimated. Some designers consider it a mistake

to thicken the walls at the junction and to tie the floor to the walls with steel;[75] they would treat the floor as a unit in itself, and waterproof the joint with asphalt, to accommodate settlement. The Bridgewater tank, reported absolutely dry, has three times as much steel at base as at Manchester; one set of rods extends 5 ft. high, one set 10 ft., one set, 15 ft., and fourth set to the top.

Observations by Andrews. Westerly standpipe leaked at the joint where the second day's work left off; the construction is shown in Fig. 236. The force diagram indicates the compressive stresses near the floor joint. The plane where compression ceases is called the critical joint. It is located by projecting the resultant backward until it intersects the face of the wall. Notice that the leaky joint is close to the critical joint. After the tank had been filled 3 weeks the leakage was just sufficient to dampen the surface. In addition to the leak near the bottom, three spots appeared: one very close to the floor, caused by a breakdown in the mixer, resulting in a delay of an hour or so; another 23 ft., and one 40 ft. up. These were grouted under high pressure and stopped. The worst of the three, before grouting, leaked one drop in 6 sec. At *Attleboro* the floor was 12 in. thick. and met the wall with a curve of radius, 5 ft. The top of the floor was reinforced with ¼-in. twisted square bars, 6-in. centers each way, carried well up the curved corner, and into the wall; ⅜-in. square twisted bars were also placed radially at intervals of about 3 ft. around the circumference, the ends projecting up into the wall 10 ft. The foundation slab, 18 in. thick, was increased immediately under the walls to 4 ft. for a width of 5 ft. A concrete curb 3 ft. high, 12 in. thick, with a curved top, was built around the outside of the standpipe bottom, but not monolithic with it. Rods at the base of *Manchester* standpipe are 1 in. diam., 12 in. on centers, and extend into the wall about 4 ft. 6 in. Seepage developed at several horizontal joints, but the upper joints gradually tightened. Examination of reservoir after emptying showed horizontal cracks at joints mentioned, about 30 ft. long, extending through the wall; also vertical cracks in the plastering extending upward 20 ft.; furthermore, there were checks in the plastering from which water oozed back into the reservoir after it was emptied. From these observations were drawn the following deductions: (1) not enough vertical steel properly disposed to distribute fully the bending moment and shearing stress between the rigid base and the walls; (2) the ultimate strength of the concrete was probably exceeded when the reservoir was filled, thus producing vertical cracks; these vertical cracks allowed the water to permeate the walls to the lines of least resistance, the horizontal joints; (3) a rich plaster coat on more or less permeable concrete was useless, as the usual crazing and the vertical cracking would allow percolation of water through it.

At Bridgewater the floor ran straight across to the outside with just a little offset downward and then a roughing strip. Then the wall was cast, dovetailed into the floor, and a little fillet of concrete was placed between the floor and wall and worked in with great care.

Sliding joint at Jones Street reservoir, San Francisco,[76] was designed to provide for expansion and contraction as well as mobility under earthquake shock. This joint has proved satisfactory and permits no leakage; the cracks so common at the base of reinforced-concrete standpipe walls are lacking. A raised rim inside the joint limits wall movement along the bottom slab in case

of earthquake. The base slab was poured first, and the area which was to support the wall was smoothed with a cement-mortar finish. To prevent adhesion between the concrete base slab and concrete wall, a coating of asphalt paint was next applied. After wall was built, a V-notch existed between the wall and the rim. To give a tight and elastic joint, this notch was calked with oakum and the upper space filled with liquid asphalt. Slip joint (patented) used at Fulton[74] consisted of placing the bottom slab first, spreading a graphite paste under the wall; sheet copper of wall width was laid on this immediately and the wall concrete started.

Leakage and Waterproofing. Preceding 1911, little attention was given to membraneous waterproofing, as watertightness was thought attainable by proper proportioning and mixing. Leakage through joints between days' work, through cracks formed by overstressing the concrete in tension or at junction of side walls and floor led to diverse preventive measures. Milinowski[40] says, "An absolutely watertight tank has practically never been achieved." However, this claim has been made for the Bridgewater tank. Tank at Westerly showed a small line of dampness at the horizontal joint about 3.5 ft. above the floor. Leaks soon developed at construction joints, and more leaks developed every time the tank was emptied and filled. Unless very low stresses are used in the hoops, or vertical steel bars are added, there may be a distinct movement on some joint, as is indicated by the fact that when a standpipe has been kept full of water for some time leakage through these joints almost entirely disappears, but, on emptying and refilling, the leakage occurs as vigorously as at first. Experience at Attleboro is that cracks under constant head tend to tighten, while those under a great variation lack this opportunity. At Manchester, the floor was finished with 1-in. granolithic, and the walls plastered two coats, making about 1 in. of 1:1 cement mortar. There developed at several horizontal joints, especially three lower joints, between days' work, some seepage of water, which in three places increased to positive leaks. Seepage at upper joints gradually stopped, presumably due to filling of pores by hydrated lime. Seepage spots show more prominently when the humidity is high. Sunny sides of standpipes have developed most cracks and leaks.

Manchester, Mass. (R. C. Allen, Engineer), standpipe leaked only on about 20 ft. on the easterly and southeasterly sides, where maximum temperature stresses occur from the sun, while in every other place there was not a drop. The leakage between the first and third joints was estimated 15,000 gal. maximum in 24 hr. That was the most serious leak; the leaks at other times have been matters of perhaps a few hundred gallons a day. In 1911 not more than 10 or 15 gal. per day was seeping out in the space 8 ft. long and 6 in. wide at the very base.

Spalling disfigures the exterior and leads to apprehension in the lay mind for the safety of the tank. At Manchester[62] frost threw off a piece of concrete about 8 ft. square, which disintegration had reduced to sand. Efflorescence rather than freezing was blamed for the unsightly effects on the Manchester tank. Exterior spalling at Attleboro became so unsightly that the standpipe was enclosed in an 8-in. brick wall. Spalling at Waltham exposed the reinforcement in spots.

Leakage prevention demands dense concrete, watertight joints between days' work, stressing the concrete only within its ultimate strength so that cracks will not form, and application of waterproofing. As building forms and placing steel preclude continuous placing of concrete, make short intervals between successive layers and bond in best possible manner. So-called waterproofing materials have been frequently incorporated in the concrete; experience has shown them to be of doubtful benefit; hydrated lime has increased efflorescence and consequent disfigurement. Effectual aids to prevention of leakage are: skilful selection and gradation of aggregates to secure maximum density; liberal use of cement; abundance of mortar; sufficient water to make concrete wet enough to flow into and completely fill small spaces where stiff concrete cannot be (or is not likely to be) forced by tamping; very thorough mixing; tight forms; skilful jogging of all concrete as deposited; keeping thoroughly wet from time forms are removed until concrete is two weeks or more old; protecting new concrete from hot sun; using cement which develops minimum temperature during setting and hardening; thorough removal of laitance, dead cement, and dirt, especially at joints; scrupulous inspection. (See also p. 799.)

Membranes and waterproof coatings have grown in favor, since they have proved the most effective remedies for leaky standpipes. Allen[62] would apply membrane waterproofing to all tanks over 50 ft. high, and would protect the outside from the elements. An interior coat of $7\frac{1}{2}$ per cent. solution of calcium fluorosilicate, followed after 24 hr. by two coats of glutin painted on, produced a tight oil tank at Mt. Hope, Canal Zone.[69] At Merrimac[68] leaky spots were waterproofed successfully with materials furnished by Arco Co., Cleveland.

Prevention of Joint Leakage. To prevent leakage through horizontal joints between pourings of different days, grooves can be formed at close of a pour to receive the section above. A better method is to use copper strips 4 to 6 in. wide, bent at right angles on both edges $\frac{1}{2}$ in., and with a small V-crimp in the center, pressed into the fresh concrete one-half their width, with end joints lapped 3 to 6 in. so as to form continuous strips around the tank. Flat steel strips about $\frac{1}{4}$ by 6 in. have also been used successfully. If properly embedded in concrete, well made, metal will probably last indefinitely.

Watertightness was secured in tanks for Atlanta Warehouse Co.,[79] by using a dense gravel concrete (1:1.5:3) and applying two coats of water-gas tar. Construction joints were made tight by 18- $\times$ $\frac{1}{4}$-in. steel plates punched along outer edge to increase bond. Construction joints every 13 ft. in 143-ft. height. Gravel did not exceed $\frac{3}{4}$ in. The first coat of waterproofing was thin, and served as a primer. No evidence of leakage after a year. Spiral stairs rather than a ladder were used, attached to the wall outside.

Repairing Leaky Standpipes. At Westerly, hot paraffin waterproofing did not produce a tight tank; plastic slate stopped some leakage but imparted a bad taste to the water. Finally, the whole inside was covered with five layers of felt saturated with "Neponsit compound," at cost of \$1782; this proved successful. At Manchester, attempts were made to repair cracks by covering with lead plates, but eventually an asphalt waterproofing was the only satisfactory remedy. At Lexington an unsuccessful attempt at water-

proofing by plastering on a lining of canvas resulted in the accumulation of the canvas in the bottom of the standpipe.

Ambursen Co.[70] repaired a tank 75 ft. in diam. and 30 ft. high, which had leaked nine years. During this time two layers of membrane waterproofing had been applied to the inside without improving conditions. The last remedy was: the interior walls were dried by salamanders and then coated with a hot elastic asphalt on which a three-ply felt was applied with asphalt mopped between the layers, and the final surface mopped with asphalt. A curtain of wire mesh was next hung covering the asphalt, and 1 in. of gunite was then shot on. The tank was found to be watertight on filling.

Waltham Standpipe.[62] When built in 1906, hydrated lime equal to 5 per cent. of weight of cement was added, and concrete was carefully proportioned. The first 10 ft. of the inside was plastered with mortar containing an integral waterproofing and the remainder received at least two brush coats of a similar mixture. In 1914 a coat of pitch was applied to the inside, and a subsequent attempt was made applying cement by the cement gun on the outside. Neither proved satisfactory; the seepage continued, and successive frost action caused several large outside areas to disintegrate to the extent of exposing the reinforcement. In 1922 the inside was relined with four layers of waterproofing felt mopped on with Texaco No. 56 asphalt, and finished with 6-oz. saturated duck fabric mopped with asphalt. Inside of this was laid up a 4-in. brick wall, the ½-in. annular space between the brick and the duck being completely filled with mortar.

Repairs to Attleboro Standpipe.[78] When built, this standpipe was given 13 coats of Sylvester wash at bottom and 9 at top; results were unsatisfactory. Some minor repairs were attempted by grouting behind a plastered face. A tough elastic membrane of 12 separate and distinct layers of felt in hot asphaltic compound was applied under a 5-year guarantee, at a cost of $3000. To lessen the effects of the spalling of the concrete exterior, an 8-in. brick casing was placed with a 2-in. annular air space between it and exterior of standpipe, to equalize temperature. All spalled spots were filled with cement mortar. Drain holes were left near base of casing, to lead off leakage before freezing. "No further trouble" up to 1924.

Architectural treatment is important to disguise leakage and spalling, which are most noticeable on plain concrete. Visible moisture on a tank creates uneasiness as to its safety. The 110-ft. standpipe in Kansas City has 12 pilasters for ornament. Appearances are emphasized in Germany (*vide* Hockenheim and Stromeyersdorf designs in *E. R.*, July 10, 1915, p. 49). At Merrimac,[68] the outside surface was wetted on removal of forms and rubbed with a carborundum block to present a uniform appearance, after which a thin brush coat of cement grout was applied, and the surface again rubbed with carborundum, followed by a dry brush.

REINFORCED-CONCRETE TANKS ON TOWERS

Design Methods. See paper of S. R. Ross in *Can. Eng.*, May 1, 1923, p. 445.

Concrete proportions used by Weston & Sampson[82] at Middleborough, Mass., were: foundations, 1:2.5:5; tower, 1:2:4; hemispherical bottom and

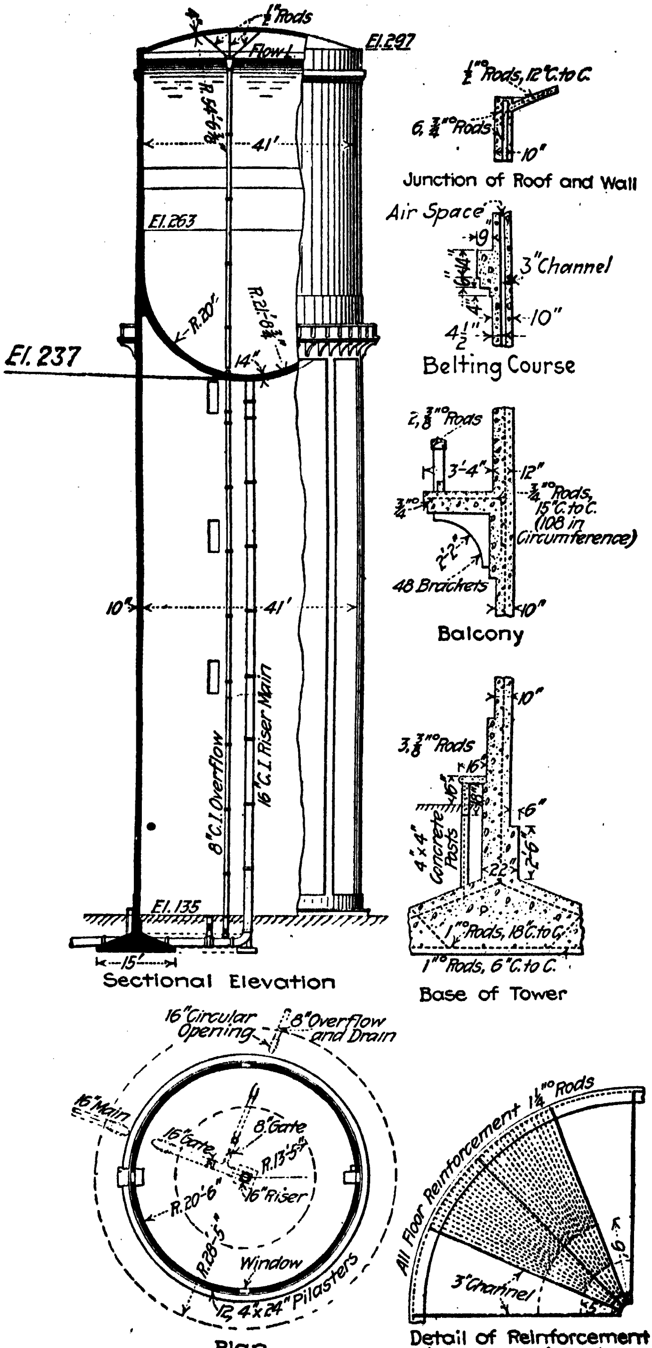


FIG. 237.—Middleborough concrete tank on tower.[82]

wall of tank up to 43 ft. above low point of hemispherical bottom, 1:1:2; for the next 14 ft., 1:1.5:3; and above, including roof, 1:2:4.

Support. For Middleborough tank,[82] a cylindrical wall was preferred to columns, for ease of calculation of stresses; could be given a better appearance by adding 12 pilasters, 24 by 4 in.; was better adapted to the contour of site; and more economical.

Wind load was figured at 30 lb. per sq. ft. of vertical projection, resulting from a 70-mi. per hr. gale.

STEEL STANDPIPES*

Large Steel Standpipes. Standpipe at Chicopee, Mass., 50 ft. in diam, 130 ft. high, capacity 2,000,000 gal., built in 1919, was subject of controversy as to excessive stresses (see *E. N. R.*, Jan. 29, 1920, p. 246).† Standpipe at Youngstown, Ohio,[85] built in 1914, 100 ft. in diam., 50 ft. high, has a capacity of 2,938,000 gal.; one at Los Angeles, 150 ft. in diam. and 51 ft. high.

Dimensions. Where a tank can be located on a natural elevation, it is economical to make diam. 10 to 20 per cent. greater than height; as a rule, height should not exceed 60 or 70 ft.

Failure of Steel Standpipes. A tank 90 ft. in diam., 50 ft. high, built in 1916 in Boston to store molasses, failed in 1919, when nearly full. Molasses weighs about 90 lb. per cu. ft. Court findings[87] were that tank shell and rivets were highly overstressed. Calculations indicate shell stress 26,000 and 36,000 lb. per sq. in.; on a net section, assuming lap-joint efficiency at 66 per cent., these become respectively 40,000 and 50,000 lb. Rivets were overstressed 2.2 times in shear, and 1.92 times in bearing.

Weakness may develop at manholes‡ unless reinforced, as these are in lowest plates, the region of heaviest load. They are generally 20 in. diam. in form of a flanged steel nozzle with a steel-plate cover bolted on.[88] Reinforcing plate should be sufficient to replace the metal cut out, and there should be enough rivets to develop full strength.

Foundations. Standpipes should be firmly set on concrete foundations and secured with anchor bolts not less than $1\frac{1}{4}$ in. in diam., set deep enough to resist uplift, with anchor plates not less than $\frac{1}{2}$ in. thick. All anchor bolts should be connected directly to sides of standpipe with bent plates or similar details. Unit stress in anchor bolts should not exceed 15,000 lb. per sq. in. of net area.

Necessity for anchorage can be determined from following equations:[90]

$$W \overset{=}{\gtrless} pkH^2,$$

in which W = total weight of metal in standpipe, lb.; p = intensity of wind pressure on normal surface, lb. per sq. ft.; k is reduction factor, to be used on account of curvature of surface; H = total height of standpipe, ft. This formula can be easily derived, on assumption that anchorage is needed when tension on windward side, due to overturning effect of wind, is equal to com-

* Among makers are: Chicago Bridge & Iron Works; Petroleum Iron Works, Sharon, Pa.; Pittsburgh-Des Moines Steel Co., New York; Tippett & Wood, Phillipsburg, N. J.; Walsh's Holyoke Steam Boiler Works; Kennicott Co., Chicago; Memphis Steel Construction Co., Greensburg, Pa.

† Constructors, Walsh's Holyoke Steam Boiler Works, report January, 1925, that tank is in excellent condition; has never had repairs.

‡ Why not omit manholes in very big or very large diam. tanks? (EDITORS.)

Table 166. Steel Standpipe Data

Diam ft	Outside circumference of inner course, ft	Bottom or cross-section, area, sq ft	Height, ft	Capacity, million gallons	Required thickness of lowest plate, unit stress not exceeding 10,000 lbs, in *	Thickness of bottom in	Approximate weight of steel, tons	Weight on foundation, tank full: Total tons	Weight on foundation, tank full: Tons per sq ft	Rivet data, vertical seams (a): Diam, in	Rivet data: Pitch, in	Rivet data: Distance between gage lines, in	Rivet data: Type of joint and assumed efficiency, %	Rivet data: Distance of gage line from edge, in	Paint, 2 field coats (b): Circumferential area, sq ft	Paint: Gals of graphite at 350 sq ft per gal	Paint: Gals of asphaltum at 310 sq ft per gal
20	62 96	314	20	0 047	1/4	1/4	9	205	0 65	5/8	1 5/8		Single lap 50	1	1260	4	4
20	"	"	30	0 071	1/4	1/4	12	307	0 98	5/8	1 5/8		Single lap 50	1	1890	5	5
20	"	"	40	0 094	1/4	1/4	16	409	1 30	5/8	1 5/8		Single lap 50	1	2520	7	8
20	63 00	"	50	0 118	5/16	1/4	20	511	1 63	5/8	1 3/4		Single lap 50	1 1/8	3150	9	10
25	78 67	491	20	0 074	1/4	1/4	12	319	0 65	5/8	1 5/8	.	Single lap 50	1	1570	4	5
25	"	"	30	0 110	1/4	1/4	16	476	0 97	5/8	1 5/8		Single lap 50	1	2360	6	7
25	78 70	"	40	0 147	5/16	1/4	21	635	1 29	5/8	1 3/4		Single lap 50	1 1/8	3150	9	10
25	78 74	"	50	0 184	3/8	1/4	26	793	1 62	3/4	2 5/8	2 1/8	Double lap 60	1 1/4	3830	11	12
25	78 77	"	60	0 221	7/16	3/8	34	954	1 94	3/4	2 3/4	2 1/4	Double lap 60	1 3/8	4720	13	15
25	78 80	"	70	0 258	1/2	3/8	42	1116	2 27	7/8	2 7/8	2 3/8	Double lap 60	1 1/2	5420	15	17
30	94 38	707	20	0 105	1/4	1/4	15	457	0 65	5/8	1 5/8		Single lap 50	1	1890	5	6
30	"	"	30	0 158	1/4	1/4	20	683	0 97	5/8	1 5/8		Single lap 50	1	2930	8	10
30	94 41	"	40	0 211	5/16	1/4	26	910	1 29	5/8	1 3/4		Single lap 50	1 1/8	3770	11	12
30	94 47	"	50	0 264	7/16	3/8	37	1142	1 62	3/4	2 3/4	2 1/4	Double lap 60	1 3/8	4720	13	15
30	94 51	"	60	0 316	1/2	3/8	47	1372	1 94	7/8	2 7/8	2 3/8	Double lap 60	1 1/2	5670	16	18
30	94 54	"	70	0 369	9/16	3/8	59	1605	2 27	7/8	2 7/8	2 3/8	Double butt 70	1 5/8	6620	19	21
30	94 58	"	80	0 422	5/8	3/8	72	1839	2 60	7/8	2 7/8	2 3/8	Double butt 70	1 5/8	7570	22	24
30	94 64	"	90	0 475	3/4	1/2	89	2077	2 94	1	3	2 1/2	Double butt 70	1 3/4	8520	24	28
30	94 67	"	100	0 528	13/16	1/2	105	2314	3 27	1	3	2 1/2	Double butt 70	1 7/8	9460	27	31
30	94 71	"	110	0 580	7/8	1	124	2554	3 61	1 1/8	3	2 1/2	Double butt 70	2	10410	30	34
30	94 74	"	120	0 633	15/16	1/2	144	2795	3 95	1 1/8	3	2 1/2	Double butt 70	2	11360	32	37
35	110 08	962	20	0 144	1/4	1/4	18	619	0 64	5/8	1 5/8		Single lap 50	1	2200	6	7
35	110 12	"	30	0 215	5/16	1/4	25	927	0 96	5/8	2 5/8	2 1/8	Double lap 60	1 1/8	3310	9	11
35	110 15	"	40	0 287	3/8	3/8	37	1240	1 29	3/4	2 5/8	2 1/8	Double lap 60	1 1/4	4410	13	14
35	110 22	"	50	0 359	1/2	3/8	48	1551	1 61	7/8	2 7/8	2 3/8	Double lap 60	1 1/2	5510	16	18
35	110 25	"	60	0 431	9/16	3/8	61	1865	1 94	7/8	2 7/8	2 3/8	Double butt 70	1 5/8	6610	19	21
35	110 32	"	70	0 502	11/16	1/2	80	2185	2 27	1	2 7/8	2 3/8	Double butt 70	1 3/4	7710	22	25
35	110 35	"	80	0 574	3/4	1/2	98	2503	2 60	1	3	2 1/2	Double butt 70	1 3/4	8830	25	28
35	110 42	"	90	0 646	7/8	1/2	118	2824	2 94	1 1/8	3	2 1/2	Double butt 70	2	9930	28	32
35	110 45	"	100	0 718	15/16	1/2	141	3148	3 27	1 1/8	3	2 1/2	Double butt 70	2	11040	31	35
35	110 51	"	110	0 790	1 1/16	1/2	166	3473	3 61	—	—	— — —	— — — — — — — —	— — —	12140	35	39
35	110 54	"	120	0 862	1 1/8	5/8	195	3803	3 95	—	—	— — —	— — — — — — — —	— — —	13240	38	43
35	110 57	"	130	0 934	1 3/16	5/8	225	4134	4 30	—	—	— — —	— — — — — — — —	— — —	14350	41	46
40	125 79	1257	20	0 188	1/4	1/4	21	807	0 64	5/8	1 5/8		Single lap 50	1	2520	7	8
40	125 82	"	30	0 282	5/16	1/4	30	1208	0 96	5/8	2 5/8	2 1/8	Double lap 60	1 1/8	3780	11	12
40	125 89	"	40	0 376	7/16	3/8	45	1616	1 29	3/4	2 3/4	2 1/4	Double lap 60	1 3/8	5090	14	16
40	125 95	"	50	0 470	9/16	3/8	60	2024	1 61	7/8	2 7/8	2 3/8	Double butt 70	1 5/8	6300	18	20
40	125 98		60	0 564	5/8	3/8	77	2434	1 94	7/8	2 7/8	2 3/8	Double butt 70	1 5/8	7560	22	24

* This cylinder formula, 1/4 in is thinnest plate to be considered in "Steel Mill Buildings." from Pencoyd Handbook (a) Johnson's "Framed Structures" based on Watertown tests (b) Quoted by Ketchum

pression due to weight of pipe. When anchor bolts are required, they should be spaced equally around circumference. Stress, lb., in an anchor bolt may be determined from equation:

$$S = \left(\frac{W - pkH^2}{D'}\right)\frac{1}{n},$$

in which D' = diam. of anchor bolt circle, ft.; n = number of equally spaced anchor bolts. This equation is readily obtained by considering standpipe as cantilever supported by anchor bolts.

For connection of shell with flat bottoms, use angle irons having thickness equal, at least, to that of bottom ring of shell. Wherever triple riveting is necessary for lower horizontal joints, two bottom angles should be used, with thickness of each angle not less than two-thirds of thickness of bottom ring. Steel standpipes may rest on either a sand cushion or a grouted base, to assure bearing being taken by bottom plates rather than rivet heads. At Needham, Mass.,[91] base was provided with 2-in. grout holes about 10 ft. apart, and a 1:1 grout of consistence of cream was poured in under 2-ft. head. Grouting holes should be placed in the field, rather than arbitrarily drilled in the shop. Grouting under low pressure may not diffuse the grout, and grouting under too high pressure may warp the base plates. Wheeler has successfully grouted under conical bottoms, with 2-ft. dip in a 40-ft. tank. This shape can best resist high pressures. On earth or other yielding foundation, if a standpipe is enclosed by a masonry tower, the two foundations should be separated by a joint permitting independent movement. This is especially desirable because of change of load as the standpipe is filled and emptied.

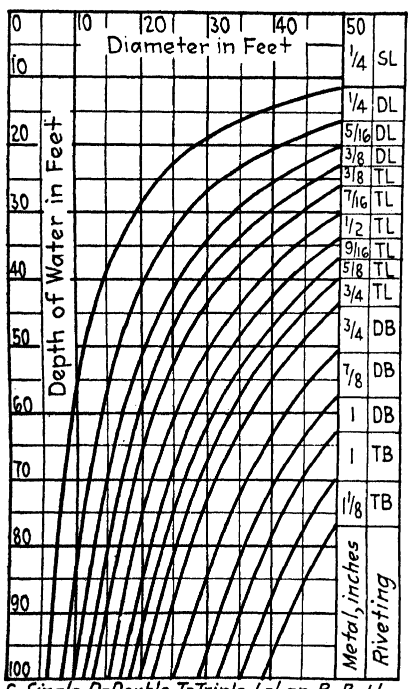


FIG. 238.—Plate thickness for cylindrical steel tanks and standpipes.[83]

Steel Plates. Minimum thickness should be $\frac{1}{4}$ in. for sides and $\frac{5}{16}$ in. for bottom. Figure 238 gives plate thickness based on steel stress of 15,000 lb. per sq. in., and minimum thickness of metal of $\frac{1}{4}$ in.; "depth of water" is measured to the lowest point of zone considered. On Fig. 238, find point of intersection of depth and diam. lines. This point will generally fall between two curves; follow space upward and to right, and read thickness of metal and type of riveted joint to be used at that depth. Keeping on the same ordinate for diam., ascertain thicknesses for other depths and change the design accordingly.[83]

Architectural Treatment. There is always opposition to bare steel standpipes because of ugliness and depreciation of real estate values. Cincinnati Water Department built concrete shells around steel standpipes. Four tanks at Eastern Hills were interconnected by concrete diaphragms to give the effect of a giant chessman.[92] The Mt. Auburn tanks, 40 ft. in diam., 70 ft. high, for many years an eyesore, were likewise encased in concrete. The cost of architectural embellishment and of parking the surrounding grounds is considered a good investment. Instead of declining, property values in the vicinity have increased. The 2,500,000-gal. Bellevue Hill[93] tank in West Roxbury, Boston, was enclosed in a shell of rock-faced granite coursed ashlar and has a roof supported on steel trusses.

Life. Wrought-iron tank erected in 1883 at Princeton, N. J.,[94] received two coats of paint when erected, was removed in 1915 because of inadequacy; had been kept full of water, and had been repainted every 2 or 3 years. It had a flat bottom, and was supported on creosoted timbers. Inspection in 1915 showed it exceptionally well preserved; rust had not caused sufficient deterioration in the plates to be perceptible by calibration. It was reerected in 1915 at Lawrenceville, N. J. Steel standpipe enclosed in brick at Madison, Wis.,[95] removed after 30 years on account of inadequacy and location, was 12.5 ft. in diam., 60 ft. deep; bottom supported on a grillage of steel rails on 16-in. I-beams, 70 ft. above the ground. Examination during removal indicated little corrosion and a possible physical life of many times 30 years. The plates had never been cleaned and repainted. At Toledo,[96] a wrought-iron standpipe, 5 ft. in diam., 224 ft. high, built in 1873, was torn down in 1916 to make way for improvements; 37 tons of wrought iron were salvaged. Steel standpipe at Black River Falls, Wis.[97] was destroyed by progressive corrosion and ice action; painted but once in 37 years. It stored soft waters containing "much organic matter." In 1904 the maintenance of an old wrought-iron standpipe was costing Attleboro[98] $400 per year. Cleanings twice a year each netted about a ton of rust, due to the large quantity of carbon dioxide in the water. Its life was placed at 20 years.

Painting* outside presents no serious problem, but maintenance of interior requires emptying; tank can seldom be out of service long enough for proper painting. Experience epitomized from a questionnaire by Metcalf and Eddy[99] indicates that tanks should be thoroughly cleaned and painted inside at least every 4 or 5 years. Use of a sandblast for cleaning and proper application of protective coating should lengthen this interval. Tests by Kneen, of Philadelphia, with different kinds of paints indicated that only paints to give reasonably good service are red oxide of iron and red lead. Sherman[99] considers best practice represented by methods of Metropolitan Water & Sewerage Board on Bellevue Hill standpipe:

Plant consisted at first of a 20-hp. gasoline-engine-driven air compressor, a compressed-air reservoir, two lines of air hose, and two nozzles. Sandblasting was stopped in late afternoon each day, and the cleaned surfaces were painted before rusting commenced. One foreman, two painters, who also did sandblasting, and one helper could sandblast and paint an area of about 330 sq. ft. per day.

* See also "Protecting Iron and Steel Standpipes from Corrosion," by Sherman, J. N. E. W. W. A., Vol. 33, 1919, p. 272.

Later, to increase progress, a second compressor, operated by an automobile engine, was installed. Entire inside and outside, 35,650 sq. ft., was sandblasted and painted. All painting materials were furnished by the department, but were mixed by the contractor under the direction of the engineer. For inside of tank, National Lead Co.'s red lead in oil paste, litharge, and Spencer-Kellogg & Son's boiled linseed oil were used; first coat natural color, the second and third coats tinted with lampblack in oil. For outside of tank, red-lead paste, raw linseed oil, and drier were used for first coat, and for second coat, white lead, raw linseed oil, turpentine, and drier tinted with lampblack. One gallon of red-lead paint was sufficient to cover 700 sq. ft. of surface with one coat.

Sherman emphasizes the thorough cleaning of the metal and the immediate application of paint before cleaned surface has cooled and moisture has condensed upon it. It appears probable from the questionnaire that not only red lead, but several graphite and red lead paints, and perhaps certain enamel-like coatings,* will give satisfactory protection. The latter has given best protection of ship bottoms. There is a lurking, although unfounded fear of lead poisoning if the water remains in contact with red-lead paint.

Specifications (1922) of Inspection Dept., Associated Factory Mutual Fire Insurance Companies:

First or shop coat: Mix 100 lb. of red-lead paste in 2½ gal. of linseed oil, equivalent in consistence to 28 lb. dry red lead to 1 gal. of linseed oil. The red lead used must contain at least 94 per cent. of true red lead (Pb_3O_4). Paste is preferable to dry red lead.

After structure has been erected and tank made watertight, the steel must be cleaned of all corrosion and dirt, and bare spots touched up with paint of the following mixture. One coat of this paint must then be applied to the entire structure.

Second coat or first field coat: Mix 100 lb. of red-lead paste in 3 gal. of linseed oil, equivalent in consistence to 25 lb. dry red lead to 1 gal. linseed oil. The red lead used must contain at least 94 per cent. of true red lead (Pb_3O_4).

This paint may be tinted in order that the desired final color may be obtained. Paste is preferable to dry red lead.

A third coat (second field coat) of paint must be applied to the structure after the preceding coat has become thoroughly dry. This should preferably be of same mixture as second coat for inside of tank but for outside any good paint may be used.

Painting must not be done out of doors during wet or freezing weather.

STEEL TANKS ON TOWERS

Large Tanks on Towers. Both wood and steel have been used for elevated tanks, and for the towers. Largest tank on tower in the United States is surge tank for Salmon River Power Co. at Altmar, N. Y.;[101] capacity, 1,500,000 gal.; height from top of foundation, 185 ft.; diam., 50 ft.; depth, 105 ft. Tank for Louisville Water Co. has capacity of 1,200,000 gal.; height, top of foundation to top of tank, 220 ft.; diam., 50 ft.; depth, 90 ft.; 48-in. riser pipe. This suggests possibility of similar tanks on high towers for fire protection in congested districts of cities. Chicago Bridge & Iron Works states that it is prepared to construct tanks of much greater dimensions than the Louisville

* Used on tank at Reading, Mass.[100] (see p. 317).

tank; one of 2,000,000 gallons is proposed for Charleston, S. C. Five tanks, each of 1,000,000 gal., were recommended for Wheeling by J. N. Chester[102] because the hilly topography of the city made reservoirs in excavation and embankment impracticable. Life of steel tanks was figured at 25 years with ordinary care.

Specifications for steel tanks have been issued by the Inspection Dept., Associated Factory Mutual Fire Insurance Companies, and will be sent free on application. See also Birch-Nord, *T. A. S. C. E.*, Vol. 64, 1909, p. 526, and Ketchum's "Structural Engineers' Handbook," 3*d* ed., 1924, p. 461. The National Board of Fire Underwriters has also issued regulations (1915) for gravity and pressure tanks.

Rectangular tanks are prevalent in English practice. See "Tank Construction," by E. G. Beck (Emmet & Co., London, 1921).

Form of Tank Bottom. When steel was first substituted for wood, flat bottoms were used. This was uneconomical, owing to the heavy floor system required, and inaccessibility of bottom for painting. A conical form has in some instances been used, but there is nothing to recommend this type, while a few notable failures are recorded against it. The most practical forms are either hemispherical or elliptical. Elliptical bottom* has following advantages, according to makers: (*a*) no greater stresses transmitted to cylinder than in the hemispherical design; (*b*) depth is saved; (*c*) the curvature selected gives the greatest capacity for the weight of steel used, and reduces the stresses in the bottom to a minimum; (*d*) it has sufficient slope so that no sediment will remain in the tank.[103] On the other hand, there is increased cost for fabrication, due to forming to compound curvature.

Pipes. Outlet pipe and inlet pipe are generally served by one riser. Overflow piping should be separate; it should always be provided, as discharge of overflow into the air both alarms observers and in a high wind sprays the near-by residents and has been the cause of complaints. In localities where water in a standpipe may freeze, the overflow should be outside, connected by an aperture through the top plate.

Riser piping is subject to great temperature influences and should be provided with an expansion joint for warm weather, and some sort of frost protection for cold weather. For moderate conditions large diam. steel pipe is recommended; as this freezes only next to the metal wall, an interior waterway is always available. In extremely cold climates, the riser pipe is enclosed in a masonry or timber shell with an insulating air space. A heating chamber is provided, with an ordinary depot stove heater. This commonly suffices to prevent all freezing troubles at a very low cost. Steam coils are installed in the bottoms of some railway tanks for maintaining the temperature of the water.

Plates. See p. 560.

Painting. See p. 561.

Life. See p. 561.

Towers. A water tower may have three or more legs. With wooden towers it is desirable to use 12 and even more legs, if the tank is very large, because a lesser number of 12 by 12's, the size of timber most suitable, would

* Patented by Pittsburgh-Des Moines Steel Co.

not give sufficient cross-section. Most builders have now adopted the four-post steel design, as being, everything considered, the best, except for tanks of very large capacities. Stone and brick are occasionally used, but usually at a greater cost. A weakness in tank construction[104] has been poorly designed connections of steel shell to the posts of the supporting tower, which allowed eccentric loading. A number of tanks have failed by the posts staving the tank plates. A horizontal girder is commonly employed to relieve the eccentricity, but tanks with girders improperly placed have likewise failed.

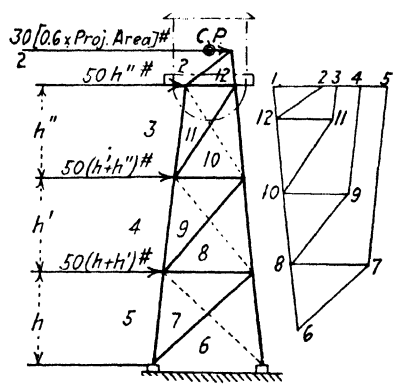


FIG. 239.—Tower design.[83]

Foundations. Average permissible pressure on soil, tons per sq. ft.: soft clay, 1; ordinary clay, 2; dry sand and dry clay, 3; hard clay, 4; gravel and coarse sand, 6. Foundations should be carried below frost, and anchor bolts placed deep enough to develop full strength. In foundations for towers with inclined legs supporting elevated tanks, care shall be taken that piers are constructed in such manner that resultant of vertical and horizontal forces, due to direct loads, passes through center of gravity of piers. Foundations, in general, should be concrete of 1 part Portland cement, 3 parts sand, and 5 parts crushed stone or gravel; where part of foundation is under water, concrete should be 1:2:4 mixture.

Table 167. Standard Steel Tank[83]

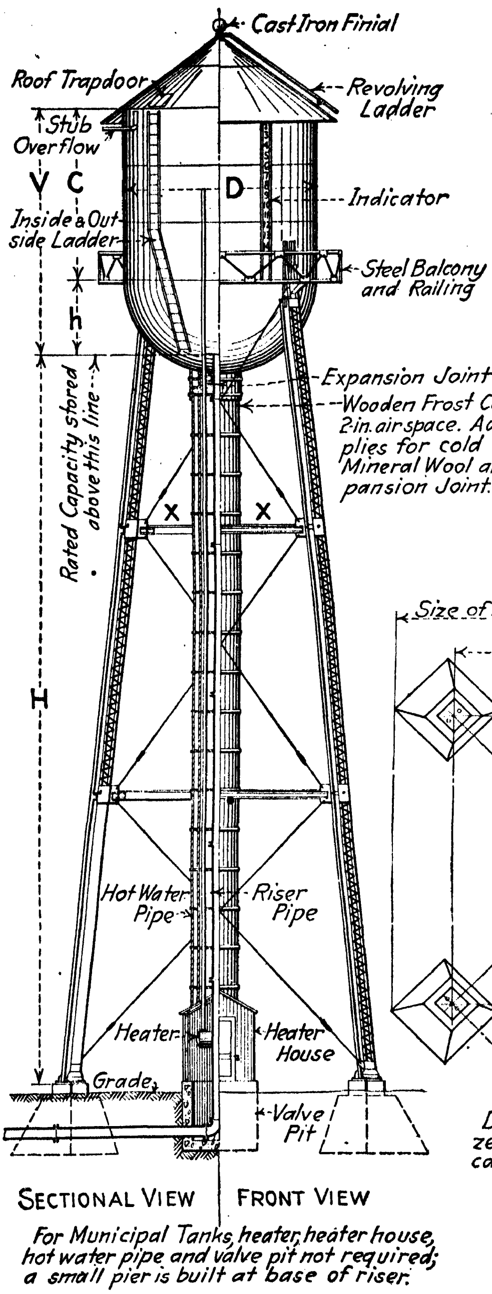
Rated capacity, U. S. gallons	Diameter "D"	Cylinder height "C"	Distance "h"	Rated capacity, U. S. gallons	Diameter "D"	Cylinder height "C"	Distance "h"
10,000	11′ 0″	10′ 11″	4′ 0″	*100,000	24′ 0″	22′ 6″	8′ 6″
15,000	13′ 0″	10′ 11″	6′ 0″	*200,000	32′ 0″	23′ 4″	12′ 9″
20,000	15′ 0″	10′ 9″	5′ 5″	250,000	32′ 0″	31′ 4″	14′ 0″
*25,000	15′ 0″	14′ 7″	5′ 4″	300,000	36′ 0″	28′ 4″	14′ 1″
*30,000	15′ 0″	18′ 5″	5′ 4″	400,000	40′ 0″	30′ 0″	16′ 6″
*40,000	17′ 0″	18′ 5″	6′ 8″	500,000	44′ 0″	29′ 9″	19′ 5″
*50,000	19′ 0″	17′ 7″	8′ 0″	600,000	44′ 0″	38′ 6″	19′ 8″
*60,000	19′ 0″	22′ 6″	7′ 3″	1,000,000	50′ 0″	52′ 0″	21′ 9″
*75,000	21′ 0″	22′ 6″	8′ 0″	1,200,000	54′ 0″	52′ 3″	25′ 3″

* Tanks carried in stock, fabricated and ready for immediate shipment. Range of head = $C + h$.

NOTE

To determine size of lot required for a specified size of tank and height of tower, take dimensions **D** *and* **h** *from table of Standard Tanks; determine* (**H** + **h**) *and apply in formula below for* **S** = *spread between adjacent column centers at base. Special Towers can be designed for restricted locations.*

For sizes of tower members and thicknesses of tank plates, send for standard plan.



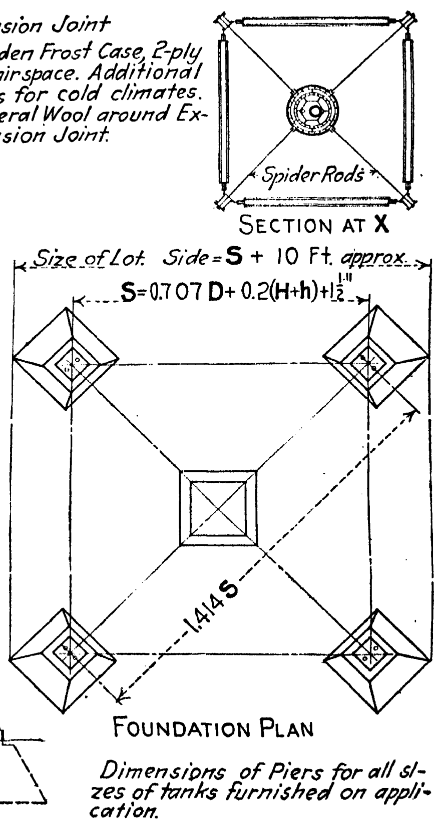
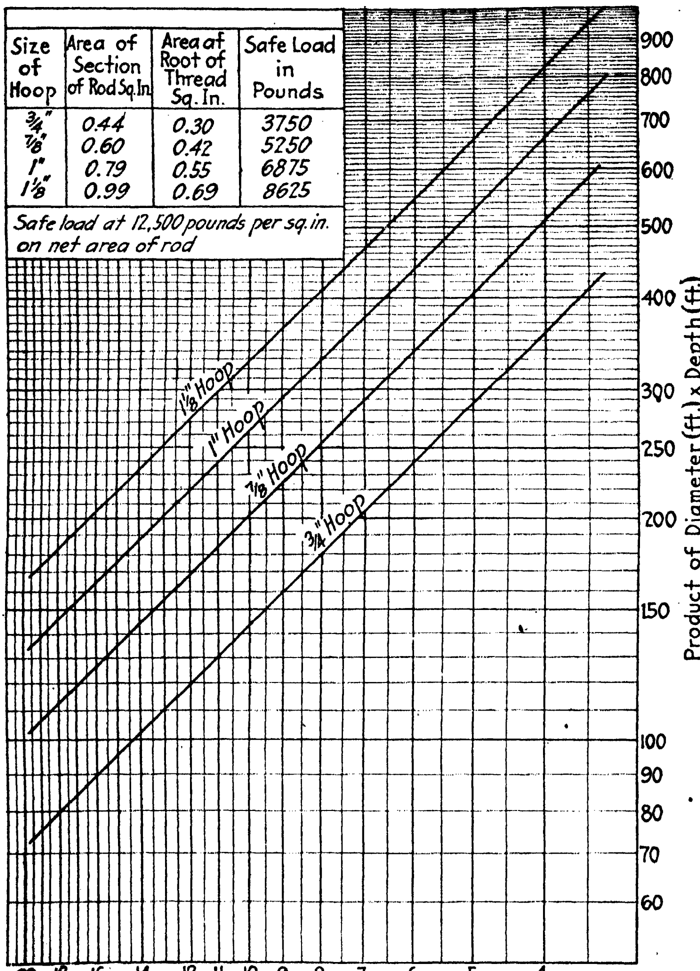


FIG. 240.—Piers for tank on tower.[83]

WOODEN TANKS*

Usefulness. Wooden tanks are in common use to supply small quantities of water on top of buildings, on low towers, or occasionally on rises of ground. Their usefulness to railroads and industries lies in their readiness of erection with cheap labor, and facility of moving to other locations. They do not lend themselves to architectural treatment. On cantonment work, redwood



Size of Hoop	Area of Section of Rod Sq. In.	Area of Root of Thread Sq. In.	Safe Load in Pounds
3/4"	0.44	0.30	3750
7/8"	0.60	0.42	5250
1"	0.79	0.55	6875
1 1/8"	0.99	0.69	8625

Safe load at 12,500 pounds per sq. in. on net area of rod

$$\text{Spacing, inches} = \frac{\text{safe load per loop, lbs.}}{2.6 \times \text{diam. ft.,} \times \text{depth, ft.}}$$

FIG. 241.—Allowable spacing of hoops on wooden tanks.[106]

tanks of 100,000- and 200,000-gal. capacity were used to conserve steel; they cost much less per gal. of capacity than either steel or concrete.[105]

Steel Tanks vs. Wooden. Gases from locomotives do not damage them and vibration of passing trains does less damage than to a steel structure, in

* Manufacturers include American Wood Pipe Co., Tacoma; W. E. Caldwell Co., Louisville; U. S. Wind & Pump Co.; National Tank & Pipe Co.

which crystallization may be provoked. Steel tanks of sizes commonly used for fire protection cost from 40 to 100 per cent. more than wooden; the additional cost of large tanks is relatively less. A steel tank of about 40,000-gal. capacity, or over, can be erected on a steel trestle at about the same cost as a wooden tank, since a saving can be made in cost of supports by a hemispherical bottom. Above 60,000-gal. capacity, Teague[104] says that steel tank is cheaper and more practicable. In large sizes, railroads always use steel. The tightness and durability of wooden tanks depends chiefly on selection of lumber and care in construction. A steel tank is superior to a wooden:[106] (1) It will last for an indefinite time if kept thoroughly painted inside and out, whereas a wooden tank will have to be replaced in from 12 to 30 years (usually 15). (2) It will be absolutely tight when once well erected and properly cared for, whereas a wooden tank will shrink and leak when the water gets low. (3) It will not be likely to burst suddenly (if originally correctly designed) even if painting is neglected, for a few spots will rust through first. Objections to steel tanks are: (1) They require skilled boiler makers to erect, thus adding considerably to the cost when at a distance from the boiler shop. (2) They

Table 168. Capacities and Weights of Small Wooden Water Tanks*
Regular Sizes from 1½- and 2-in. Lumber[107]

Length of stave, ft.	Diam. of bottom, ft.	No. hoops	Capacity, bbls.	Capacity, gals.	Approximate weight, 2-in. pine, or 1½-in. cypress, lbs.	Length of stave, ft.	Diam. of bottom, ft.	No. hoops	Capacity, bbls.	Capacity, gals.	Approximate weight, 2-in. pine, or 1½-in. cypress, lbs.
2	4	2	4	117	180	2	8	2	17	543	450
2.5	4	2	5	158	210	2.5	8	2	23	710	510
4	4	4	8	268	325	3	8	3	28	884	590
5	4	4	11	342	385	4	8	4	39	1,218	740
6	4	5	13	410	460	6	8	5	60	1,878	1,000
2	5	2	6	195	240	7	8	6	70	2,207	1,170
2.5	5	2	8	255	280	8	8	7	80	2,535	1,345
4	5	4	14	443	425	10	8	8	101	3,180	1,520
5	5	4	18	562	500	12	8	9	121	3,816	1,700
6	5	5	22	675	590						
7	5	6	25	784	690	2	9	2	22	696	645
						2.5	9	2	29	904	710
2	6	2	9	292	290	6	9	5	76	2,390	1,250
2.5	6	2	12	382	330	8	9	7	102	3,210	1,550
3	6	3	15	477	400	10	9	8	128	4,020	1,830
4	6	4	21	659	480	12	9	9	153	4,820	2,125
5	6	4	27	838	550						
						2	10	2	28	870	750
6	6	5	32	1,017	640	2.5	10	2	36	1,138	825
7	6	6	38	1,190	740	8	10	7	129	4,060	1,865
8	6	7	43	1,368	850	10	10	8	162	5,102	2,150
10	6	8	54	1,710	960	12	10	9	181	5,700	2,465
2	7	2	13	408	360	2	12	2	40	1,270	1,000
2.5	7	2	17	534	410	2.5	12	3	53	1,661	1,116
6	7	5	45	1,413	900	8	12	7	189	5,944	2,072
7	7	6	53	1,660	1,025	10	12	8	237	7,476	2,421
8	7	7	60	1,886	1,150	12	12	9	286	8,998	2,770
10	7	8	74	2,335	1,300	14	12	12	335	10,544	3,200

* See also Table 169, p. 568.

Table 169. Capacities and Weights of Standard Size Wooden Railroad and Storage Tanks[107]

Made from 3-in. Material

Length stave, ft.	Diam., ft.	No. hoops	Capacity		Weight, lbs.	
			Gals.	Bbls.	Pine	Cypress
10	10	8	4,750	150	3,100	3,700
12	10	9	5,700	182	3,500	4,300
14	10	10	6,680	212	4,000	4,800
10	12	8	7,053	224	4,000	4,800
12	12	9	8,488	269	4,500	5,600
14	12	10	9,902	314	5,200	6,300
16	12	12	11,293	358	5,800	7,100
10	14	8	9,773	310	4,800	5,900
12	14	9	11,774	374	5,500	6,700
14	14	10	13,750	436	6,200	7,600
16	14	12	15,701	498	6,900	8,500
10	16	8	12,935	410	5,700	7,000
12	16	9	15,597	495	6,700	8,200
14	16	10	18,229	579	7,300	9,000
16	16	12	20,833	661	8,200	10,000
18	16	14	23,406	743	9,200	11,200
12	18	9	19,956	633	7,700	9,300
14	18	10	23,340	741	8,700	10,600
16	18	12	26,689	847	9,700	11,800
18	18	14	30,004	952	10,900	13,200
20	18	15	33,288	1,057	11,900	14,400
12	20	9	24,852	788	9,000	10,100
14	20	10	29,080	923	10,100	12,200
16	20	12	33,270	1,056	11,200	13,600
18	20	14	37,423	1,191	12,500	15,100
20	20	15	41,540	1,319	13,700	16,500
12	22	10	30,285	961	10,300	12,500
14	22	11	35,451	1,125	11,500	13,900
16	22	12	40,576	1,288	12,600	15,300
18	22	14	45,660	1,449	14,000	17,000
20	22	16	50,702	1,609	15,400	18,600
12	24	10	36,254	1,151	11,600	14,000
14	24	11	42,453	1,347	12,900	15,600
16	24	13	48,606	1,543	14,300	17,300
18	24	14	54,714	1,737	15,700	19,000
20	24	16	60,778	1,929	17,300	20,800
14	26	11	50,071	1,589	13,800	18,100
16	26	13	57,360	1,821	15,700	19,000
18	26	14	64,587	2,050	17,200	20,800
20	26	16	71,766	2,278	19,000	22,900
16	28	14	66,785	2,121	17,500	22,700
18	28	15	75,449	2,395	18,800	24,400
14	30	12	67,150	2,132	17,300	20,800
16	30	14	77,044	2,446	19,900	23,900
18	30	15	86,790	2,755	21,500	25,800
20	30	17	96,480	3,063	23,700	28,300

are more difficult to protect against freezing. (3) They give more trouble by sweating when in a building. (4) They deteriorate rapidly if painting is neglected.

Creosoted wood tanks have proved a profitable investment in railway service.[110] Creosoted loblolly pine is standard on Illinois Central R. R.[109] for normal railway service; in high tanks, steel is used. Timber is air-seasoned for about 3 months, and is treated by the Rueping process, applying 5 lb. of oil per cu. ft.

Life.[109] Timber gives longest life when used in territory where grown. Redwood tanks have a life of 26 to 48 years in California, and 15 in Wisconsin. White pine tanks have a life of 35 years in Michigan and 13 in Missouri. One cedar tank is reported in good condition after 42 years service. Of 184 tanks

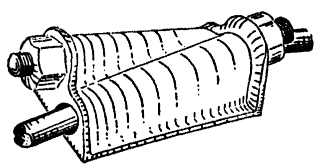


Fig. 242.—Hoop lug.

in railway service Knowles reports average life as 32.6 years for redwood, 25 to 32 years for cypress, and 30 to 35 years for white pine.

Towers may be of timber, steel, or concrete. Chicago & Northwestern Ry. found that a reinforced-concrete enclosure had same first cost as a steel tower, and eliminated maintenance cost.[111] Supports, whether tower or platform, should be unyielding, as leakage has often been traced to joints opening through settlement. An I-beam grillage is preferred for a foundation.[104]

Water-level recorders* furnish a continuous record of the stages of the reservoir, and aid in computing water-consumption rates. They must be protected from freezing and from derangement by floating objects. Distance recorders show the pumping station operator the stage of the reservoir. They require a special pole line for their electrical operation, which is apt to be too costly to install. The Bristol equipment must be operated by a.c. at about 110 volts and requires three wires. Some error is always to be expected in gages of the float type, but they are generally accurate enough for most practical purposes.[113] Telltales do not furnish a continuous record, but enable the operator to learn the stage of the tank. Telltales are liable to get out of order and give a false notion of the water stage. Teague[104] considers the mercury gage developed by the Associated Factory Mutual Fire Insurance Companies the most reliable telltale.

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* Made by Sanborn Co.; Boston; Gurley; Winslow, Waltham; Bristol Co.; and others.

14. *E. R.*, Sept. 30, 1916, p. 408. **15.** *E. R.*, Oct. 28, 1916, p. 527; *E. N.*, Aug. 5, 1915, p. 267. **16.** Hill: *Proc. A. W. W. A.*, 1902, p. 23. **17.** *E. N.*, Nov. 25, 1915, p. 1034. **18.** *E. N. R.*, May 3, 1917, p. 241. **19.** *E. N. R.*, Aug. 24, 1922, p. 309. **26.** *E. C.*, Apr. 10, 1912, p. 415. **27.** *E. N.*, Oct. 5, 1916, p. 646. **28.** *E. N.*, Dec. 23, 1915, p. 1206. **29.** *E. N. R.*, Feb. 23, 1922, p. 310. **30.** *E. N. R.*, June 2, 1921, p. 958. **31.** *E. N. R.*, Aug. 26, 1920, p. 410. **32.** Catalog of Gunite Co. **33.** *E. N. R.*, Nov. 25, 1920, p. 1044. **34.** *E. N. R.*, Feb. 10, 1921, p. 260. **35.** *E. N. R.*, Sept., 23, 1923, p. 518. **36.** *E. R.*, Nov. 29, 1913, p. 616. **37.** *E. C.*, Feb. 23, 1916, p. 186. **38.** *E. N. R.*. Apr. 11, 1918, p. 702. **39.** *E. N. R.*, July 1, 1920, p. 36. **40.** *J. A. W. W. A.*, Vol. 10, 1923, p. 575, **41.** *E. N.*, May 25, 1916, p. 990. **42.** *E. N. R.*, Nov. 15, 1917, p. 928. **43.** *E. N. R.*, July 23, 1915, p. 517. **44.** *E. N. R.*, May 4, 1922, p. 739. **45.** *E. N.*, Dec. 7, 1916, p. 1099. **46.** *E. N. R.*, March 1, 1923, p. 390. **47.** *E. N. R.*, Nov. 21, 1918, p. 933. **48.** Merrill: *J. N. E. W. W. A.*, Vol. 29, 1915, p. 152. **49.** *E. C.*, Jan. 7, 1914, p. 8. **50.** *E. N. R.*, Apr. 12, 1917, p. 85. **51.** Folwell: "Water Supply," p. 345. **52.** *E. N. R.*, May 8, 1924, p. 813. **53.** *E. N. R.*, Dec. 13, 1917, p. 1107. **54.** *E. N. R.*, Sept. 29, 1921, p. 535. **55.** *T. A. S. C. E.*, Vol. 86, 1923. **a**, p. 192; **b**, p. 182; **c**, p. 189. **56.** *E. N. R.*, Sept. 18, 1919, p. 566. **57.** *E. N.*, Feb. 5, 1914, p. 320. **58.** *E. R.*, July 29, 1916, p. 151. **59.** *E. N. R.*, May 22, 1919, p. 1014. **60.** *E. N. R.*, Oct. 2, 1919, p. 673. **61.** *E. N. R.*, Jan. 9, 1919, p. 96. **62.** *J. N. E. W. W. A.* Vol. 29, 1915, p. 184. **63.** *E. N.*, Mar. 18, 1915, p. 554. **64.** *E. N.*, Apr. 29, 1915, p. 837. **65.** *E. N. R.*, Oct. 28, 1920, p. 841. **66.** *E. C.*, May 26, 1915, p. 462. **67.** *E. N. R.*, Feb. 1, 1923, p. 217. **68.** *E. N.*, Dec. 23, 1915, p. 1226. **69.** *E. N. R.*, Feb. 19, 1920, p. 380. **70.** *E. N.*, June 1, 1916, p. 1044. **71.** *E. R.*, Feb. 17, 1916, p. 277. **72.** *E. N.*, Nov. 9, 1916, p. 894. **73.** *Cement Age*, September, 1911. **74.** *E. C.*, Apr. 22, 1914, p. 460. **75.** *E. N.*, Sept. 7, 1911, p. 298. **76.** *E. N.*, Feb. 18, 1915, p. 291. **77.** *E. N.*, May 13, 1915, p. 950. **78.** *E. N.*, Apr. 29, 1915, p. 817. **79.** *E. R.*, Mar. 31, 1917, p. 505. **82.** *E. N.*, Aug. 26, 1915, p. 392. **83.** Catalog, Pittsburgh-Des Moines Steel Co. **85.** *E. N.*, Feb. 19, 1914, p. 412. **87.** *E. N. R.*, May 15, 1919, p. 974; Jan. 29, 1925, p. 178. **88.** *E. N. R.*, May 13, 1920, p. 952. **90.** Birch-Nord: *T. A. S.-C. E.*, Vol. 64, 1909, p. 542. **91.** Sherman: *J. N. E. W. W. A.*, Vol. 30, 1916, p. 132. **92.** *E. N. R.*, Aug. 16, 1917, p. 305. **93.** *E. N.*, Jan. 20, 1916, p. 109. **94.** *E. N.*, Jan. 27, 1916, p. 150. **95.** *E. N. R.*, Mar. 31, 1921, p. 548. **96.** *E. N. R.*, Jan. 24, 1918, p. 174. **97.** *E. N. R.*, June 28, 1923, p. 1135. **98.** *E. N.*, Apr. 29, 1915, p. 816. **99.** Sherman: *J. N. E. W. W. A.*, Vol. 35, 1921, p. 51. **100.** *E. C.*, July 14, 1915, p. 32. **101.** *E. N.*, July 16, 1914, p. 151. **102.** *E. N. R.*, Feb. 7, 1924, p. 229. **103.** *E. N.*, June 25, 1914, p. 1438. **104.** Teagues: *T. A. S. M. E.*, Vol. 35, 1913, p. 921; *E. C.*, Sept. 16, 1914, p. 266. **105.** Maury: *T. A. S. C. E.*, Vol. 83, 1919–1920, p. 500. **106.** Inspection Department, Associated Factory Mutual Fire Insurance Companies, 1922. **107.** Catalog, U. S. Wind Engine & Pump Co. **109.** *E. N. R.*, Oct. 24, 1918, p. 754. **110.** *E. N. R.*, July 15, 1920, p. 113. **111.** *E. N. R.*, Dec. 9, 1920, p. 1143. **112.** *E. N. R.*, Nov. 28, 1918, p. 996. **113.** Stevens: *T. A. S. C. E.*, Vol. 83, 1919–1920, p. 894. **115.** *E. N. R.*, Jan. 17, 1918, p. 131. **116.** Catalog, Lancaster Iron Works. **117.** *E. N.*, Oct. 12, 1916, p. 693. **119.** Trans. *A. S. C. E.*, Vol. 88, 1925, p. 1274. **120.** *Mun. & Co. Eng.*, May, 1921, p. 198. **121.** *Munic. Eng.*, 1917, Vol. 53, p. 191, **122.** *E. R.*, Sept. 21, 1907, p. 312.

CHAPTER 24

WATER CONSUMPTION*

Users of water comprise domestic, agricultural, industrial, and public users. The latter two are classed as "non-domestic" by committee of A. W. W. A. Excess of any class may affect rate of consumption. The high rates in Table 172 for some western communities are due to irrigation.

Domestic requirements* depend on the standard of living of the users; committee of N. E. W. W. A.[1] found a daily rate of 15 g.p.cap. in lowest-class dwellings (13 persons per house); 34 g.p.cap. in middle-class dwellings (7 per house); and 54 g.p. cap. in first-class dwellings (6 per house). Pennsylvania Hotel, New York City, accommodating 3300 guests,† averages 0.9 mgd. For demands on iced-water systems, see *Power*, Feb. 12, 1924, p. 254. Abel Wolman[2] places "sanitary" demand at 15 to 25 gal.; 30 should constitute the "maximum water requirement." The net daily allowance in cantonment design‡ was 55 g.p. cap.; it was considered "very liberal," but was exceeded in many camps.[3] The troops in France had an average of 2 g.p.d.[4] Lawns require 7 to 8 gal. per 100 sq. ft. for sprinkling and double this quantity for soaking. Tests[5] by W. F. Sullivan on lawn sprinklers and garden-hose nozzles indicate that discharge per hr. varies from 225 to 660 gal., dependent on type and pressure. A. O. True[6] estimates water-closet flushing at 12 to 20 per cent. of total consumption; his tests of wash-down siphon closets indicate that they can operate on as little as $2\frac{1}{2}$ gal. per flush; 5 to 6 gal. is usually allowed and $1\frac{1}{2}$ gal. for ordinary urinal. Ordinary bath tub requires 30 gal. Public baths at New Orleans, when emptied daily, require 200 to 500 gals. of water per bather.[21]

Table 170. Per Capita Allowances, Gal. per Day, for Apulian Aqueduct System, Italy[8]

Type of supply	Type of community				
	Large cities: Bari, Lecce, Foggia, Barletta, Tarento	Population			
		Above 20,000	20,000–10,000	10,000–5,000	5,000 or less
Gravity......	24	18.5	16.0	14.5	13.25
Pumped......	....	....	13.25	12.0	10.6

Industrial requirements for railroads, shops, restaurants, etc. vary with character of industry and have no fixed relation to population; among the

* For Waste of Water, see pp. 423–426.
† With employees and visitors this hotel has been termed equal to a town of 10,000.
‡ For aviation fields, see *J. A. W. W. A.*, Vol. 6, 1919, p. 668.

largest users are tanneries, laundries, breweries, chemical works. The paper industry requires 0.12 to 1.8 mg. per ton of paper.[9] Figures on industrial uses of water are presented in committee reports cited below. Many industries find economy in providing their own water supply. Industrial depressions have a marked effect on consumption; in 1920, the consumption in Newark averaged 48 mgd., and in 1923, although the population was larger, an industrial depression reduced the demand to 44 mgd. For steam making, water required per hr. per hp. may be taken at 25 lb. for good engines, and 15 lb. for the best engines. Condenser water for large plants is generally taken directly from adjacent streams. Railroad centers supply a large quantity to railroads. In Watertown, N. Y., one-third of demand is industrial.[11]

Public uses of water, including waste,* often constitute a large proportion of consumption. Municipal requirements for fire fighting, street and sewer flushing, and for use in public buildings are not metered in many cities where rigorous accounting methods do not prevail; such consumption is often included in "Water not accounted for," see p. 423. For pump slip, see p. 482. Fire-protection requirements during a year are small, but a heavy demand for brief periods (for demands at some notable fires, see Wegmann's "Conveyance of Water," p. 399). The National Board of Fire Underwriters requirements are given in Table 106, p. 401. (For further discussion of public uses see *J. N. E. W. W. A.*, Vol. 27, 1913, pp. 117–134, and Caleb Mills Saville, in *J. A. W. W. A.*, Vol. 7, 1920, p. 869; also *ibid.*, 914.) Waste, often euphemistically termed "water not accounted for," may range from 20 to 50 per cent. in completely metered cities.† Tests at Rochester[19] indicated as follows: $2\frac{1}{2}$-in. hose for street flushing with 1-in. smooth noozle and 30-lb. hydrant pressure averaged 1000 gal. per 1000 sq. yd. Sewer flushing at 150 gal. per min.; time varies from 15 to 60 min. For snow removal, 100,000 gal. per day is required for 30 days. Fountain, 25 gal. per day by meter; operates 5 months per year. Horse troughs, metered flow 1,500,000 gal. per year.

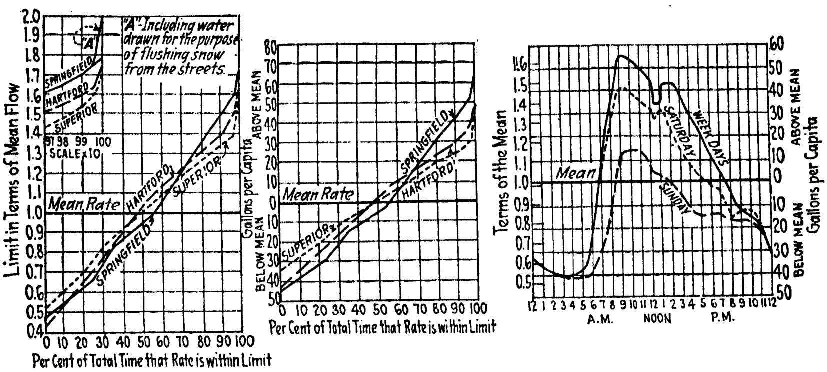
Water-consumption statistics for a selected large, small, and medium-sized community in many states are presented in Table 172. "Gal. to each consumer" represents gal. per capita. There is lack of uniformity in methods of reporting; many offices now use the N. E. W. W. A. forms to avoid this. See also Reports of Committees on Water Consumption, *J. N. E. W. W. A.*, Vol 27, 1913, p. 29, and *J. A. W. W. A.*, Vol. 2, 1915, p. 181; also "Consumption and Sales of Water in Water-works Systems that Are Completely Metered, or Nearly So," in "*Meter Rates for Water-works*," by Allen Hazen (Wiley, 1918), p. 202. For indication of increasing rate of use of water compare recent figures with those in *Report to Merchants' Assn.*, New York (1906), by J. H. Fuertes; in a *Treatise on Hydraulic and Water-supply Engineering* (1899), by J. T. Fanning; in *Report upon New York's Water Supply* (1900), by J. R. Freeman, p. 33, and in *J. N. E. W. W. A.*, Vol. 27, 1913, p. 40.

Statistics of foreign cities, Table 173, are reproduced from the first edition, as world conditions render impracticable the securing of authoritative new data. Other data on foreign cities are given in *J. N. E. W. W. A.*, Vol. 27,

* See p. 423.
† See p. 425.

1913, pp. 47–53. For English statistics, see also "Water Works Directory and Statistics," published annually by Hazell, Watson & Viney, Ltd., London. For statistics of most European cities, see Capacci, Acquedotti, ed Acque Potabile (Hoepli, Milan, 1918.)

Per capita consumption in cities and towns of the United States ranges approximately from 50 to 400 g.p.d. For communities having service con-



In terms of mean. Above and below mean. In hourly rate.

FIG. 243.—Variation of water consumption in completely metered systems.[20]

nections wholly or largely metered, it is commonly under 100 g.p.d., and for small cities and towns often much less. For large cities with few meters, but well-managed works in good condition, 125 to 150 g.p.d. is a reasonable allowance. Character of industries, climate, and other local conditions have

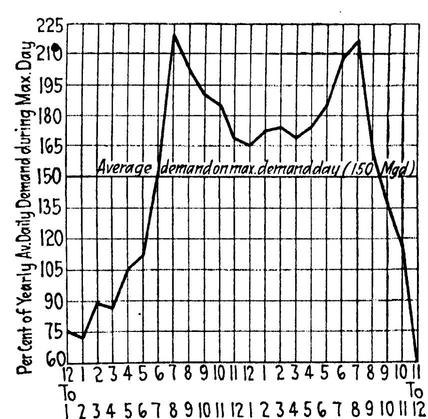


FIG. 244.—Hourly variations of pumping demand on a maximum day,* Cleveland, O.[18]

important influences. The pressure is also a factor; see Metcalf's diagram for Akron, *J. N. E. W. W. A.*, Vol. 33, 1919, p. 219.

W. R. Hill[12] tabulated records from 68 cities to show a lower consumption rate in the cities more completely metered. Metcalf studied relation of water

* Similar diagram for Detroit in J. A. W. W. A., Vol. 8, 1921, p. 600.

rates and growth of communities to consumption, see *Water Works*, April, 1926, p 157

Fluctuations in consumption rate are caused by the necessities of household activities, by fires, by allowing fixtures to run in cold weather, by

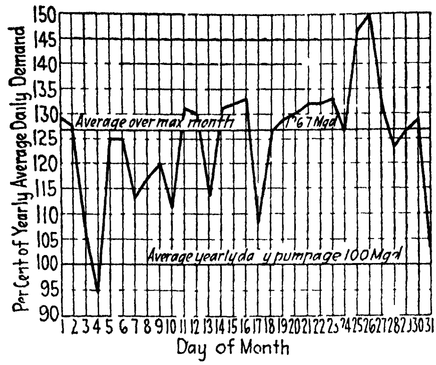


FIG 245—Variations in daily pumping demand of maximum month having maximum variations in demand in 10 year period, Cleveland[18]

excessive bathing and sprinkling in a hot spell Figure 244 shows typical variations I F Wall concludes from St Louis data[13] that the daily rate determined from annual rate will be exceeded by 125 per cent in the maximum month, by 135 per cent in the maximum week, and by 150 per cent in the

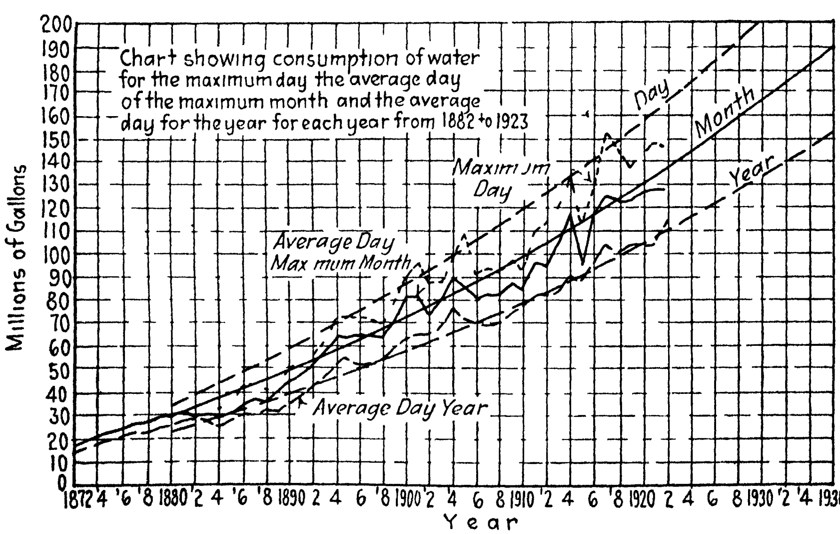


FIG 246—Consumption, St Louis, 1882 to 1923

maximum day. Large cities usually have much less proportionate variation in consumption throughout the 24 hr than small communities where fire demands have large influence Pumping records, 1914–1915, at San Juan, P.R., showed maximum daily at 125 per cent. of average, and maximum

hourly at 168 per cent.[14] Tests by Burgess at Lorain, Ohio, showed daily consumption in July, 1912, of 4.15 mgd., and rate for maximum 4 hr. averaging 6.25, 151 per cent. of July average. Standard Schedule, National Board Fire Underwriters, assumes maximum daily consumption 150 per cent. of yearly daily average.

Table 171. Night Flow vs. Daily Rate[15]

City	Population	Per cent. metered	Daily consumption per cap., gals.	Flow, 1 to 4 A.M. in per cent. of daily rate
Detroit, Mich.	1,000,000	98	135	66
Hartford, Conn.	122,000	86	70	53
Milton, Mass.	9,400	100	46	21

Forecasts of consumption must be based on the extent and character of community to be supplied. In business districts, estimates are sometimes based on floor-space area in the office buildings.[16] Studies must recognize that, even in cities 100 per cent. metered, the rate of consumption is legitimately increasing. It is often convenient to estimate separately the needs of the various classes. Hazen's studies for the future needs of northern New Jersey are based on 116 g.p.d. per capita. The increasing rates per capita assumed in Baltimore report by Stearns and Freeman[17] are as follows: year 1915, 130; 1920, 135; 1930, 145; 1940, 150 g.p.day. Freeman's 1900 report for New York assumed 128 in 1920; records show average was 132 for 1920 and 127 for 1921.

Agricultural Uses. Consumption of water by farm animals varies greatly, depending upon season of year, age and individual habits of animal, and local conditions. Following is an approximation: Horses, each, 5 to 10 gals. per day; cattle, each, 7 to 12 gals. per day; hogs, each, $1\frac{1}{2}$ to $2\frac{1}{2}$ gals. per day; sheep, each, 1 to 2 gals. per day. Making allowance for evaporation it requires 28,320 gals. to irrigate 1 acre 1 in. deep. It requires from 10 in. to 20 in. per acre to produce a crop, average being 16 in. Actual amount required depends upon crop and season. Flooding alfalfa 6 in. deep will wet the soil 4 ft. from surface. Flooding orchards 4 in. deep will wet the soil 4 ft. from surface.

Table 199. Daily Consumption of Water by Crops
(Risler)

	Inches		Inches
Lucern	0.134 to 0.267	Wheat	0.106 to 0.110
Meadow grass	0.122 to 0.287	Rye	0.091
Oats	0.140 to 0.193	Potatoes	0.038 to 0.055
Corn	0.111 to 1.570	Oak trees	0.038 to 0.050
Clover	0.140	Fir trees	0.020 to 0.043
Vineyard	0.035 to 0.031		

Meters may cut consumption in irrigated districts as at Twin Falls, Idaho.[22]

Table 172. Water Consumption in American Cities

Secured Mainly by Correspondence Supplemented by Data in "American City" Jan., 1921, pp. 41–49

State and municipality	Population served, thousands	Gals. per day to each consumer	Metered: Per cent. of receipts	Metered: Per cent. of service	Ave. consumption, mgd.	Year
Alabama						
Birmingham	238	55	91.4	99.7	13.0	1922
Eufaula	6	77	....	60	0.5	1920
Mobile	69	150	....	66	10.3	1922
Arizona						
Bisbee	18	22*	100	100	0.4	1922
Prescott	8	63	....	100	0.5	1920
Tucson	25	200	....	44	5	1923
Arkansas						
Helena	12	100	....	78	1.2	1920
Little Rock	85	82	....	100	7.0	1923
Pine Bluff	15	80	....	100	1.2	1920
California						
Palo Alto	7	107	....	100	0.8	1920
Pasadena	57	106	94	....	6.1	1921
San Diego	121	87	100	100	10.5	1923
Colorado						
Colorado Springs	35	191	26	2	6.7	1922
Denver	290	192	20	2	55.9	1922
Sterling	6	116	100	100	0.7	1923
Connecticut						
Hartford	165	84	....	96.7	13.9	1922
Middletown	16	96	....	99	1.5	1920
New London	25	104	100	....	2.6	1921
Delaware						
Georgetown	2	25	....	2	0.05	1920
Wilmington	112	112	61.8	88	12.5	1922
Florida						
Jacksonville	92	69	....	98	6.3	1920
Lakeland	10	50	....	98	0.5	1923
Quincy	5	19	100	20	0.1	1923
Georgia						
Atlanta	275	89	....	100	24.6	1923
Augusta	55	90	....	100	5.0	1922
Savannah	85	123	50	25	10.5	1923
Idaho						
Boise	30	50	92	100	1.5	1922
Coeur d'Alene	8	316	....	Small	2.5	1922
Lewiston	10	70	95	87	0.7	1923
Illinois						
Bushnell	3	33	....	100	0.1	1920
Chicago	2938	276	58	....	785	1921
Peoria	100	88	71	81	8.8	1922
Indiana						
Evansville	85	99	75	....	8.4	1922
Ft. Wayne	87	66	....	77	5.7	1920
New Albany	23	109	....	9	2.5	1920
Iowa						
Council Bluffs	38	118	100	100	4.5	1923
Davenport	56	84	70	88	4.8	1922
Oskaloosa	10	100	95	90	1	1923
Kansas						
Concordia	....	52	100	100	5	1922–3
Kansas City	110	138	79	90	15.2	1922

* Unusual local conditions.

Table 172. Water Consumption in American Cities.—(*Continued*)

State and municipality	Population served, thousands	Gals. per day to each consumer	Metered: Per cent. of receipts	Metered: Per cent. of service	Ave. consumption, mgd.	Year
Kentucky						
Ashland	22	58	77	93	1.3	1922
Covington	70	71	90	100	5	1922
Louisville	258	138	....	11.7	35.9	1922
Louisana						
Baton Rouge	20	88	95	90	1.8	1920
New Orleans	405	100	....	100*	40.2	1922
Shreveport	44	100	....	81	4.4	1920
Maine						
Augusta	13	154	....	....	2	1923
Bath	12	125	35	30	1.5	1923
Waterville	13	252	....	2	3.4	1920
Maryland						
Baltimore	760	124	....	....	94.5	1922
Chestertown	2.5	64	....	25	0..2	1923
Hagerstown	30	120	94	90	3.6	1923
Massachusetts						
Boston	752	125	....	100	94.3	1920
Cambridge	114	96	....	47	11.1	1922
Malden	49	55	....	97.4	2.7	1920
New Bedford	135	80	100	94.7	10.8	1922
Michigan						
Detroit	1063	144	97.7	....	153	1923
Grand Rapids	150	107	99	99	16	1923
Jackson	48	94	100	100	4.5	1923
Minnesota						
Duluth	100	89	70	79	8.90	1922
New Ulm	7	43	....	100	0.3	1922
St. Paul	241	85	95	93	20.7	1922
Mississippi						
Jackson	30	128	72	67	3.9	1923
Meridian	28	125	98	98	3.5	1923
New Albany	3	33	....	88	0.1	1920
Missouri						
Kansas City	334	133	....	80	....	1921
Poplar Bluff	8	100	....	6	0.8	1920
St. Louis	786	132	42	....	103.5	1922
Montana						
Bozeman	8	282	....	10	2.1	1922
Butte	60	146	42	5.7	8.8	1922
Lewistown	5	400	25	10.6	2	1922
Nebraska						
Grand Island	14	251	....	100	3.5	1920
Lincoln	60	90	99	100	5.4	1923
Sidney	3	100	97	95	0.3	1923
Nevada						
Elko	2	241	....	56	0.5	1920
Reno-Sparks	16	670†	....	0.1	10.8	1923
New Hampshire						
Concord	20	110	87	70	2.2	1923
Keene	11	100	95	98	1.1	1923
Portsmouth	14	65	100	100	0.9	1923
New Jersey						
Kearny	30	300	....	100	9	1923
Newark	415	92	....	93	39	1921
Paterson	136	81	....	....	11	1921

* 68 per cent., consumption. † Irrigation uses are included.

Table 172. Water Consumption in American Cities.—(*Continued*)

State and municipality	Population served, thousands	Gals. per day to each consumer	Metered: Per cent. of receipts	Metered: Per cent. of service	Ave. consumption, mgd.	Year
New Mexico						
Carlsbad	2.5	100	....	96	0.25	1920
Deming	3	68	80	77	0.22	1923
Tucumcari	4	19	....	90	0.07	1923
New York						
Buffalo	515	237	46.5	....	120.7	1921
Johnstown	13	184	....	10	2.3	1922
Syracuse	180	160	....	97	25.7	1921
North Carolina						
Concord	10	75	100	96	0.8	1922
Greensboro	25	100	90	75	2.5	1923
Wilmington	39	85	....	39	3.3	1923
North Dakota						
Carrington	1.5	22	....	100	0.03	1920
Fargo	25	100	80	100	2.5	1923
Minot	6.5	92	100	100	....	1922
Ohio:						
Cincinnati	404	118	92.7	....	47.8	1921
Cleveland	950	135	....	98	128.3	1921
Toledo	243	104	....	97	25.2	1920
Oklahoma						
Blackwell	12	63	....	99	0.75	1920
Kiefer	2	111	....	67	0.20	1920
Oklahoma City	....	75	100	100	9	1923
Oregon						
Eugene	12	104	....	100	1.3	1922
Klamath Falls	6	167	....	67	1.0	1920
Portland	285	111	52.8	31	31.6	1921
Pennsylvania						
Harrisburg	76	131	....	86	9.9	1921
Philadelphia	1823	173	....	23	315	1921
Reading	110	148	73.3	....	16.2	1921
Rhode Island						
Pawtucket	....	....	....	95	9.5	1922
Providence	276	82	....	95	22.6	1922
Woonsocket	50.3	54	99	98	2.7	1922
South Carolina						
Charleston	68	97	....	98	6.6	1920
Columbia	45	140	....	100	6	1923
Spartanburg	30	70	100	100	2.1	1923
South Dakota						
Mitchell	8.5	59	....	100	0.5	1920
Sioux Falls	33	77	97	97	2.5	1922
Watertown	10.0	50	....	29	0.5	1920
Tennessee						
Knoxville	77.8	11	....	100	0.9	1920
Memphis	175	73	97	100	12.8	1922
Murfreesboro	5.5	82	....	60	0.45	1920
Texas						
Austin	40	120	100	100	4.8	1923
Port Arthur	22.2	20	....	100	0.5	1920
San Antonio	200	115	55	51	23	1922
Utah:						
Salt Lake City	118	220	....	32	26	1920
Vermont						
Barre	10	....	43	45	....	1922
Bristol	1.2	75	....	2	0.09	1920

Table 172. Water Consumption in American Cities.—(*Concluded*)

State and municipality	Population served, thousands	Gals. per day to each consumer	Metered: Per cent. of receipts	Metered: Per cent. of service	Ave. consumption, mgd.	Year
Vermont (*Cont.*)						
Burlington	23	60	92	99	1.4	1923
Montpelier	8	300	....	....	2.4	1923
Virginia						
Alexandria	18	89	....	39	1.6	1920
Lynchburg	40	114	69	50	4.6	1922
Richmond	185	95	99	88	17.6	1922
Washington						
Centralia	9	333	20	12	3.0	1922
Seattle	350	106	84	100	37	1922
Spokane	106	220	95	95	23.3	1921
West Virginia						
Charleston	55	164	....	75	9.0	1923
Clarksburg	30	120	60	31	3.6	1923
Wheeling	54	297	....	9	16.2	1920
Wisconsin						
Madison	45	80	99	....	4.0	1921
Milwaukee	500	134	96	....	66.9	1920
Racine	63	94	88	100	5.90	1922
Wyoming						
Casper	32	109	100	99.8	3.5	1923
Cheyenne	18	400*	....	....	....	1923
Sheridan	10	175	80	90	1.8	1923

* No restrictions to date; gravity supply.

Table 172A. Industrial and Commercial Use of Water in Typical American Industrial Cities*

City	Consumption, mgd.: Total	Consumption, mgd.: Industrial and commercial	Per cent. industrial
Akron, Ohio	16.6	2.5	15
Baltimore, Md	105.0	26.3	25
Bridgeport, Conn	23.0	12.0	52
Chicago, Ill	806.0	156.0	19
Kansas City, Mo	50.0	17.5	35
Milwaukee, Wis	69.0	45.0	65
New York City	800.0	175.0	22
Rochester, N. Y	26.0	6.7	26
Springfield, Mass	13.9	4.2	30

* From "Manual of American Water Works Practice," 1925, p. 430.

Table 173. Water Consumption in Foreign Cities

Information mainly from correspondence, July, 1913

City	Population supplied	Gals. per day per capita	Metered, % of consumption	Year	Consumption, mgd.	Remarks
Canada						
Montreal	600,000	153(*a*)	10.0	1912	80.4	(*a*) Municipal; 30% supply from private sources, 104 gal. per capita per day.
Toronto	425,000	118	22.5	1911	50.3	
Vancouver	125,000	173	---	1912	21.6	All but domestic supplies metered.
Winnipeg	225,000	56	---	1912	---	Large % metered.
South America						
Buenos Aires	1,252,000	35*	100.0	1913	43.3	
Lima	175,000	57	None	1913	10.0	Estimated.
Montevideo	363,000	11	100.0	1913	4.1	Private company.
Rio Janeiro	1,000,000	60	10.0	1912	59.8	
Sao Paulo	332,000	63	---	1912	21.2	50% services metered.
Great Britain						
Belfast	400,000	48	20.0	1912	19.2	
Birmingham	852,000	32	33.5	1913	26.9	
Dublin	307,000	46	22.0	1909	14.1	
Edinburgh	450,000	56	25.0	1912	25.2	
Glasgow	1,135,000	76	27.6	1913	86.5	Water area extends beyond city.
Liverpool	914,000	42	30.0	1912	38.8	Exclusive of 24 mgd. from private sources.
London	6,677,000	43†	---	1913	28.4	23.3% of revenue comes from metered services.
Manchester	1,400,000	36‡	40.0	1913	50.4	Water area extends beyond city.
Northern Europe	(North of	50° Lat.)				
Amsterdam	566,000	26	100.0	1911	15.0	
Berlin	2,063,000	35	---	1911	70.0	25% supplied privately; 92% of public supply metered.
Brussels	312,000	25	100.0	1911	7.8	
Copenhagen	476,000	33	45.3	1913	15.6	
Dresden	555,000	30	72.4	1911	16.6	
Hague	292,000	20	50.0	1912	5.9	Some private supplies.
Hamburg	977,000	37	97.3	1912	36.0	
Leipsig	599,000	19	87.0	1912	11.4	
Moscow	----	---	100.0	1911	18.5	
Petrograd	2,018,000	38	86.5	1913	76.0	
Rotterdam	441,000	29	27.5	1912	12.6	Many private industrial supplies.
Stockholm	376,000	27	52.0	1912	10.0	
Warsaw	750,000	25	87.0	1912	18.6	
Southern Europe						
Athens	188,000	34	None	1913	6.3	Supply inadequate.
Budapest	910,000	58	39.3	1912	53.2	
Geneva	131,000	217	83.0	1912	28.4	
Madrid	570,000	84	25.0	1913	47.8	Of which 3% is private.

* 33 in 1899 and 72 in 1923 (Pub. Hlth. Eng. Abs., June 21, 1924).
† 43 in 1923 and 1924
‡ 40 in 1919 (maximum). 32 in 1922 and 1924.

Table 173. Water Consumption in Foreign Cities.—(*Continued*)

City	Population supplied	Gals per day per capita	Metered, % of consumption	Year	Consumption, mgd.	Remarks
Southern Europe						
Marseilles	550,000	44	---	1912	24 5	70 large meters. Small private supplies.
Munich........	615,000	75	---	1912	46.0	20%, private. 92% of public supply is metered.
Odessa..........	580,000	19	80 0	1912	11 0	
Paris...........	3,430,000	38	100 0	1911	111.0	Eng. News, Aug. 24, 1911.
Rome........	542,000	120	100 0	1911	65 0	
Venice..	132,000	40	65 0	1912	5 3	
Vienna	2,065,000	25	50.0	1911	51 3	
Africa and Australia						
Alexandria . ..	420,000	28	50 0	1913	11.9	Private works. Natives take water from hydrants.
Cairo	705,000	25	---	1912	16 9	Population estimated.
Melbourne.....	604,000	73	---	1912	44 4	30.7% of services metered.
Sydney.......	731,000	50	---	1912	36 4	20 7% of services metered.
Asia						
Bombay.......	979,000	37	---	1913	36 2	Some private supplies. 18 2% services metered.
Calcutta......	1,109,000	62	2 0	1913	68 1	
Canton........	1,000,000	---	Small	1913	5 6	Used by only portion of population.
Osaka	1,148,000	17	---	1912	19 7	
Shanghai..	488,000	---	9 3	1912	11 2	Population, 1910 French settlement supplied by private works, 14.13% of services metered.
Tokio	1,447,000	32	27 1	1912	46 0	

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Table 173A.—Records of Maximum Water Consumption for Massachusetts Cities and Towns, 1910*

(By courtesy of X. H. Goodnough)

City or town	Population	Average daily consumption per person	Max. monthly consumption		Max. weekly consumption		Max. daily consumption	
			Gal. per person per day	Per cent. of average for year	Gal. per person per day	Per cent. of average for year	Gal. per person per day	Per cent. of average for year
Abington and Rockland	12,383	45	63	137	70	151	90	197
Amesbury	9,894	44	50	114	51	116	68	155
Andover	7,301	86	99	115	......	......	162	189
Attleborough	16,215	54	62	115	63	116	91	169
Avon	2,013	36	55	153	72	200	102	283
Ayer	2,797	50	64	128	72	144	172	342
Beverly	18,650	91	146	160	191	210	224	246
Braintree	8,066	81	87	107	93	115	108	133
Bridgewater and E. Bridgewater.	11,051	22	27	123	29	132	39	177
Brockton	56,878	39	45	115	55	141	69	177
Brookline	27,792	89	103	116	117	132	128	177
Cambridge	104,839	100	106	106	111	111	119	119
Canton	4,797	61	75	123	84	138	97	159
Danvers and Middleton	10,536	89	108	121	136	153	158	178
Dedham	9,284	129	153	119	163	128	182	141
Easton	5,139	24	23	117	38	158	63	263
Fall River	119,295	44	47	107	50	114	54	123
Foxborough	3,863	50	51	102	59	118	75	150
Framingham	12,948	48	58	121	65	137	86	179
Franklin	5,641	61	88	144	96	158	127	208
Gardner	14,699	44	49	111	55	125	108	246
Gloucester	24,398	55	96	156	114	207	130	237
Grafton	5,705	18	22	122	24	133	'32	178
Hudson	6,743	49	60	123	65	133	......	......
Ipswich	5,777	42	60	143	84	200	106	253
Lawrence	85,892	45	51	113	60	133	60	133
Lowell	106,294	51	57	112	66	129	75	147
Lynn and Saugus	97,383	72	79	110	87	121	108	150
Manchester	2,673	120	261	217	327	271	363	302
Mansfield	5,183	75	97	129	103	137	276	368
Marblehead	7,338	79	147	186	169	214	187	237
Marlborough	14,579	37	42	114	59	159	80	190
Maynard	6,390	36	39	108	47	130	60	167
Methuen	11,448	38	54	142	67	176	69	182
Middleborough	8,214	42	53	126	65	155	90	214
Milford and Hopedale	15,243	51	60	118	64	125	71	139
Montague and Erving	8,014	66	75	114	70	106	153	232
Nantucket	2,962	67	128	191	154	230	176	263
Natick	9,866	57	70	123	82	144	170	298
Needham	5,026	66	88	133	98	148	119	180
New Bedford	96,652	81	88	109	98	121	106	131
Newburyport	14,949	68	83	122	94	138	121	178
Newton	39,806	63	74	118	82	130	95	150
North Andover	5,529	40	53	132	64	160	78	195
North Attleborough	9,562	52	73	140	82	158	95	183
North Brookfield	3,075	66	81	123	112	170	213	319
Norwood	8,014	63	86	136	86	136	132	211
Orange	5,282	26	34	131	41	158	62	182
Peabody	15,721	168	198	118	182	108	270	161
Plymouth	12,141	103	131	127	140	136	171	166
Provincetown	4,369	38	69	182	77	203	93	245
Randolph and Holbrook	7,117	74	120	162	140	189	175	237
Reading	5,818	35	52	149	60	172	66	189
Rockport	4,211	72	148	205	196	272	212	295
Salem	43,697	90	101	112	103	114	133	148
Sharon	2,310	57	97	170	120	210	137	240
Stoughton	6,316	35	43	123	58	166	75	214
Taunton	34,259	63	70	111	74	118	87	138
Wakefield	11,404	61	85	139	107	175	127	208
Walpole	4,892	102	119	117	149	146	252	247
Waltham	27,834	88	95	108	98	111	108	123
Webster	11,509	38	49	129	53	139	72	189
Wellesley	5,413	61	68	111	79	129	112	184
Whitman	7,292	29	42	145	44	152	......	......
Winchendon	5,678	30	35	117	40	133	45	150
Woburn	15,308	139	172	124	190	137	232	167
Worcester	145,986	74	85	115	......	......	103	139
Average	......	63	81	128	93	147	123	198

* Metcalf and Eddy, "American Sewage Practice," Vol. 1, p. 176.

PART V

CHARACTER AND TREATMENT OF WATER

CHAPTER 25

CHARACTER OF WATER

GENERAL PROPERTIES

Properties of Pure Water. Pure water is composed of 2 atoms of hydrogen and 1 of oxygen (H_2O) It is a liquid having a blue color, and it dissolves most substances, even rocks and metals, to a greater or lesser degree It is nearly incompressible Freezing point 0°C (32°F) Boiling point 100°C. (212°F)

Table 174. Densities and Volumes of Water from 0 to 100°C.
At Atmospheric Pressure = 760 Mm Mercury[1]

Temp t°C	Weight of 1 c c water in grams	Volume of 1 g water in c c	Temp t°C	Weight of 1 c c water in grams	Volume of 1 g water in c c	Temp t°C	Weight of 1 c c water in grams	Volume of 1 g water in c c
0	0 99987	1 00013	20	0 99823	1 00177	50	0 98807	1 01207
4	1 00000	1 00000	30	0 99568	1 00434	80	0 97183	1 02899
10	0 99973	1 00027	40	0 99225	1 00782	100	0 95838	1 04343

Table 175. Solubilities of Various Gases in Pure Water at Various Temperatures and Normal Pressure (760 Mm.)
C c per Liter

Temp Cent	Temp Fahr	Carbon dioxide[3]	Oxygen[2]	Nitrogen[2]	Hydrogen sulfide[4]	Oxygen from atmospheric air[5]	Chlorine[6]
0	32	1713	49 24	23 00	4621	10 26	
5	41	1424	43 21	20 64	3935	9 02	
10	50	1194	38 37	18 54	3362	8 02	3095
15	59	1019	34 55	16 84	2913	7 21	2635
20	68	878	31 44	15 54	2554	6 50	2260
25	77	759	28 90	14 43	2257	6 00	1985
30	86	665	26 65	13 55	2014	...	1769

Water dissolves from air relatively larger proportions of carbon dioxide than of oxygen, and of oxygen relatively larger proportions than of nitrogen, notwithstanding the ratio of N O in the air = 4 1

Conversion of c.c. per L. to mg. per L. may be effected by the following formula:

$$\frac{V}{2W} = \text{mg. per L. when}$$

V = c.c. per L.
W = weight of 1 L. of the gas, in g. (Table 176)
2 = molecular factor for bivalent gases, all gases in Tables 175 and 176 are bivalent

Table 176.* Weight of 1 Liter of Various Gases at 0°C. and 760 Mm. Pressure

Name of gas	Formula	Weight, grams	Specific gravity, air = 1
Carbon dioxide	CO_2	1.977	1.529
Nitrogen	N_2	1.251	0.967
Oxygen	O_2	1.429	1.105
Hydrogen sulfide	H_2S	1.538	1.189
Air		1.293	1.000
Hydrogen	H_2	0.08987	0.0695

* Values taken from Smithsonian Inst. Tables.

Absorption of Gases from Air by Water. On account of partial pressure being exerted, water does not absorb as much of any one gas from the air as when pure gas is brought in contact with water. The volumes of oxygen absorbed from the air by water at different temperatures are given in Table 175. The volume of any gas dissolved by water at constant temperature varies directly with the pressure.

Absorption of Light by Water.—Dr. Birge states that at a depth of 1 meter the solar energy varies in different lakes from 2 to 20 per cent. of that at the surface. The intensity of light does not decrease geometrically as the depth increases arithmetically because the water is blue in color and absorbs red and yellow rays most readily.

EXAMINATION OF WATER

An analytical examination of water consists of a series of tests and experiments, usually from 10 to 25 in number, to assist in ascertaining its past history and its present condition. For the former purpose, chemical examination is of the most value; for the latter, bacteriological; the value of these is increased many fold if accompanied by a careful sanitary inspection of the source of the sample. The important object of a water analysis is the determination of the presence or absence of unpurified human, animal, or industrial wastes, particularly sewage. Therefore, in the language of the late Professor Kinnicut: "It is very obvious that no single set of chemical standards will be safe to use in commending or condemning a given water." The opinion of the analyst must be based upon the analytical results considered in the light of the geological and sanitary conditions revealed by the inspection of the sources. For methods of physical, chemical, bacteriological, and microscopic tests, see "Standard Methods of Water Analysis," American Public Health Assn., 1925, abstracted on pages 593–601.

The following synopsis explains in brief the significance of the various tests. For a more technical explanation, see Am. Pub. Health Assn., "Standard Methods of Water Analysis," 1925; Mason, "Examination of Water," 5th ed., 1922; Woodman and Norton, "Air and Food," 1915; Ohlmüller-Spitta, "Untersuchung des Wassers," 1910; J. C. Thresh, "The Examination of Water and Water Supplies," 1904; G. C. Whipple, "Microscopy of Drinking Water," 1921; Prescott and Winslow, "Elements of Water Bacteriology," 1924; Kossowicz, "Einfuhrung in die Mykologie der Gebrauchs und Abwasser," 1913.

Physical and Chemical Examination. *Color* is a measure of colored substances in solution, such as vegetable matter dissolved from roots, leaves, and swamps, also humus and iron salts, expressed in terms of standard Pt-Co solution (see p. 594).

Odor is a measure of the odor produced by the various substances contained in the water, whether vegetable matter in solution, microscopic organisms, or gases of decomposition. Odor is useful in detecting the presence of sewage. The microscopical examination is invaluable for determining causes of odors in surface water. Many of the causes are obscure and need further study.

Turbidity is a measure of suspended matter which obstructs the passage of light. Turbidity may be due to silt, clay, suspended Fe, organic matter, micro-organisms, etc. It is expressed in terms of the turbidity produced by a given weight of silica.

Oxygen consumed is a measure of the carbonaceous organic matter which is partly oxidized by acid $KMnO_4$ solution. Waters which oxidize rapidly usually contain unstable carbonaceous matter. Oxygen consumed is closely related to color.

Nitrogen exists in water in several states of combination. Organic nitrogen in oxidizing to nitrates, passes through an intermediate stage, nitrites, or may be decomposed with evolution of NH_3 or free N. "A state of change is a state of danger" (Drown).

N as albuminoid ammonia is an approximate measure of the nitrogenous organic matter. The method determines but part (about one-half) of the total organic nitrogen. It is from two sources—vegetable and animal. Proteids and amino bodies from vegetable sources are much more stable than those from sewage and evolve N less rapidly when treated with alkaline $KMnO_4$ and distilled. Vegetable albuminoid ammonia is accompanied by color and may be due in part to presence of microscopic organisms, algæ, Crenothrix, etc. These are in suspension; therefore, the importance of determining suspended albuminoid ammonia. In potable surface waters, albuminoid ammonia should not exceed 0.3 p.p.m., although waters may safely contain more. Pure ground water contains very little albuminoid ammonia, and those containing 0.15 p.p.m. or more are usually condemned.

N as free ammonia is a result of decomposition of organic nitrogen and usually indicates roughly the amount of comparatively recently decomposed material. More than 0.15 p.p.m. must be regarded with suspicion. Free ammonia in deep wells is often of fossil origin and has little sanitary significance.

Nitrites are an indication that either oxidation of organic nitrogen or decomposition of nitrates is taking place. Their presence in any considerable amount in drinking water must always be regarded with suspicion, although deep well waters of great purity often contain nitrites.

Nitrates are a measure of completely mineralized nitrogen and when present in considerable amounts are usually indicative of past contamination.

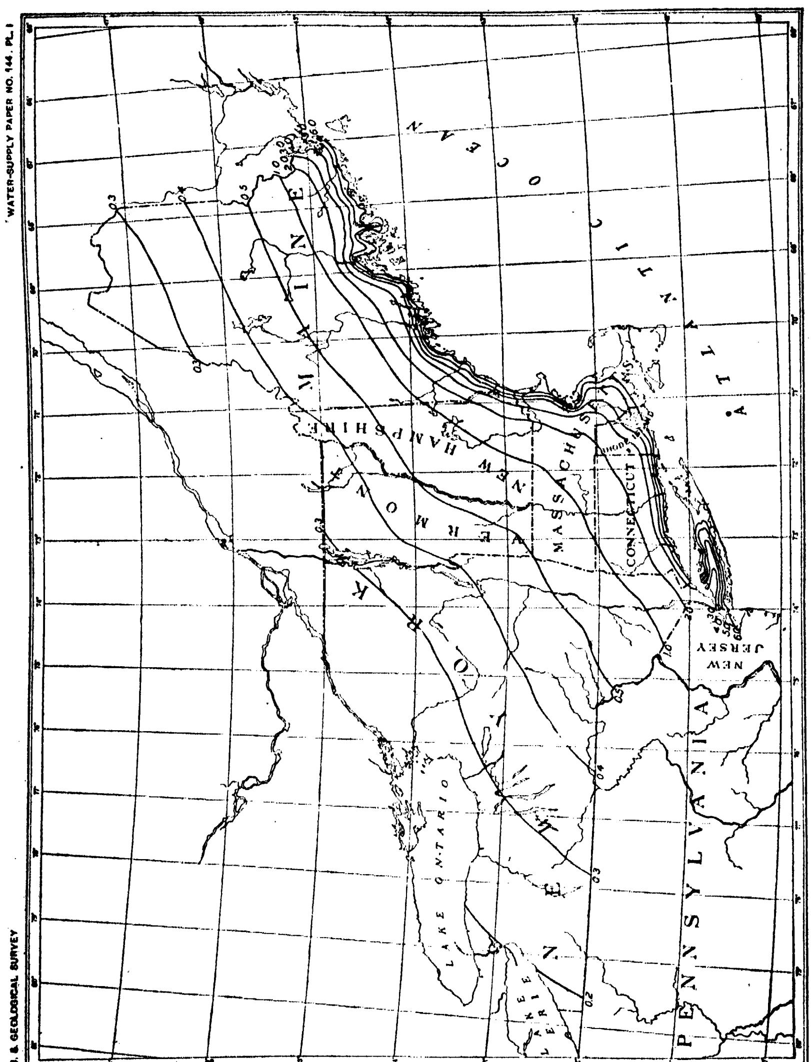

Fig. 247.—Normal chlorine map of New England and New York.

Alkalinity is a measure of the carbonates, bicarbonates, and hydroxides, and occasionally the borates, silicates, and phosphates present.

Total hardness by soap is a measure of the soap-consuming power of the water expressed in terms of $CaCO_3$. It is a measure of the Ca and Mg salts present and includes both *alkalinity* and *mineral-acid hardness*, which latter is due to presence of chlorides, sulphates, and nitrates. It may be

determined most accurately by calculation (Ca $\times$ 2.497 + Mg. $\times$ 4.115); also roughly by the soap method. Mineral-acid hardness is often called *permanent hardness* or *incrustants;* it is not removed by boiling.

Chlorine is found in all natural waters. Its source may be salt deposits in the soil. Chlorine is carried inland from the sea by the air currents and precipitated with the rain. Where the normal chlorine is known, determination of chlorine is of great sanitary significance. An excess, in absence of salt deposits, is a sure indication of present or past pollution. A chlorine map has been prepared for the Eastern portion of the United States (see Fig. 247). In localities where salt deposits affect the water, determination of normal chlorine is impracticable.

Total residue is a measure of the matters dissolved and suspended in the water.

Poisonous Metals. Service pipes and metal vessels in contact with water often dissolve readily. This is usually due to the presence of oxygen assisted by carbon dioxide and by the absence of mineral constituents which would retard the action. Poisoning by copper, zinc, and lead, most frequently by the latter metal, is likely to occur. A water containing as low as 0.5 p.p.m. of lead has caused lead poisoning,* and even smaller amounts of any of these metals have been known to endanger health when taken regularly.

Hydrogen-ion Concentration: an expression of the intensity factor of acid or alkaline properties as opposed to the quantity factors, "acidity" and "alkalinity." Usually expressed in terms of *pH* value, which is equivalent to the negative logarithm of the number of mols of ionized hydrogen per L. For water analysis the *pH* value may be determined by comparison with indicators whose colors in solution are characteristic of the hydrogen-ion concentration in solution.†

Micro-organisms, according to custom, include all organisms which are microscopic or partly visible to the eye, except the bacteria mentioned elsewhere. *Microscopic organisms* do not require culture media and may be examined with an ordinary microscope using 10 $\times$ 1 ocular and 16 mm. objective. Detailed methods for identifying and enumerating organisms are given in the following: Whipple, "Microscopy of Drinking Water," 1920; Am. Pub. Health Assn., "Standard Methods," 1925; "Aquatic Microscopy," Stokes, 1918; "The Yeasts," Guilliermond-Tanner, 1919; "Iron Bacteria," Ellis, 1923; "Fresh Water Biology," by Ward and Whipple, 1918; and "Microbiology," by Marshall, 1912. In brief, tests consist in concentrating 500 c.c. or less of the water by filtration through sand; suspending the sand in 5 c.c. of water, 1 c.c. of which is placed in a special counting cell 1 mm. deep; and examining under the optical combination mentioned, equipped with an eyepiece micrometer so graduated that it covers 1 sq. micron of the cell on the stage of the microscope. For field work Bunker has devised a sling filter and a simpler circular cell. Usually 10 or 20 squares are counted. Masses of zoöglea and amorphous material, also the organisms themselves, are counted as standard units by most observers. One standard unit = 400 sq. microns; 1 micron (μ) = 0.001 mm. The sand used is washed and screened between 60- and 100-mesh sieves. If possible, observations

* Also called "Plumbism."

† See "Standard Methods," A. P. H. A. 1925; also Clark "Determination of Hydrogen-ion Concentration," (Williams and Wilkins, 1924). Colorimetric standards may be purchased in sets, with directions for use, from dealers in laboratory supplies.

should be made immediately after collection, as many microscopic organisms, especially colonial forms, are so fragile that they disintegrate rapidly.

Plankton (from the Greek word πλαηκτος, meaning "that which wanders") is determined by filtering a large volume of water in a Plankton net or filter, weighing or measuring the residue. Results are expressed as milligrams per L. Plankton includes all suspended organisms, including bacteria. For methods and classification, see "General Bacteriology," Jordan, 1925; "Water Supplies," Savage, 1906; "Elements of Water Bacteriology," Prescott and Winslow, 1924; "Untersuchung des Wassers," Ohlmüller-Spitta, 1910, "Fresh Water Biology" Whipple-Ward (Wiley, 1918), and other treatises.

Fungi and Molds. Fungi are flowerless plants without chlorophyl or starch. They are either saprophytic or parasitic. Some authorities class bacteria among fungi, thus relating them to mushrooms. Fungi consist of a vegetative portion called mycelium, which forms the ground work of common mold or mildew; and the fruit, which is borne at the ends of the branching filaments and consists of spore cells, sometimes highly colored. Well-known molds are Mucor, Penicillium, and Aspergillus. In investigations of water supplies, the most important fungi are Crenothrix and other so-called iron bacteria,* which grow abundantly in waters containing iron and manganese. These organisms have the power of precipitating iron or manganese upon the exterior of their filaments, forming a sheath of oxide. Sometimes the growth of these fungi and the precipitation of iron or manganese is so great as to clog completely services and small mains, and either abandonment or deferrization of the supply may be necessary. Dying iron bacteria decompose, give off disagreeable odors, and may impart bad tastes to the water.

Bacteriological examination† for certain unicellular forms is important. Certain of these bacteria are normal inhabitants of all surface waters and some ground waters. Some bacteria are entirely harmless, some very harmful, and some indicative of possible presence of harmful forms. Each species has both its natural and its accidental habitat. Any examination of water does not reveal all the bacteria present; methods for this have not been discovered, but the number growing under standard conditions is an index of the amount of pollution or contamination and sometimes of the quantity of food material available for their growth. Under normal conditions a well water should contain fewer than a surface water, and surface water with sewage contamination should show increased numbers. Fluctuations in heights of streams and general weather conditions have a marked effect upon numbers of bacteria in surface waters.

Of far greater importance, however, than mere numbers is the detection of certain organisms known to accompany fecal pollution. Many normal water bacteria are unable to grow at 37°C., while almost all fecal organisms grow at this temperature. Unfortunately, many harmless soil bacteria are able to grow at 37°C. (98°F.). According to Savage,[6]

Water only at 37°C. (98°F.) showed 3 bacteria per c.c.
Water only at 21°C. (70°F.) showed 76 bacteria per c.c.
Water + soil at 37°C. showed 1630 bacteria per c.c.
Water + soil at 21°C. showed 1970 bacteria per c.c.

* See "Iron Bacteria," by Ellis (Methuen, Co., Ltd., 1923).
† See also Suter and Wells, *J. A. W. W. A.*, Vol. 9, 1922, p. 502.

In 1903, Boston sewage tested at M. I. T.[33] Sanitary Research Station showed 5,430,000 bacteria per c.c. when grown on gelatin at 20°C.; 3,760,000 per c.c. when grown on agar at 37°C. In the latter case, 1,670,000 were acid formers.*

Bacteria are valuable indicators of the present condition of any water. The total number is of value in making routine tests from a single source, though liable to insignificant variations. Ordinary procedure enumerates only a part of the bacteria present. The proportion enumerated is variable. Of much more value is the determination of the members of the group of bacteria, of which *Bact. coli* is the type. Presence of these indicate pollution and possible presence of pathogenic bacteria, such as *B. typhosus* and *B. paratyphosus.* A search for *B. enteriditis* or *Welchii*[8] is often of value.

Numbers of bacteria are determined by mixing 1 c.c. of sample of water diluted if necessary with 10 c.c. of Std. melted gelatine or agar on culture plates (Petri dishes) and counting the colonies appearing on the solidified medium after proper incubation. Dilutions must be made with sterile water. Samples must be well shaken before dilution or plating. Place 1 c.c. of water in Petri dish, add 10 c.c. of media, mix, solidify culture, and incubate. Dilutions should be planned to have from 30 to 300 colonies on plates for counting. Gelatin plates are incubated for 48 hr. at 20°C.; agar plates for 24 hr. at 37.5°C. Counting should be made by using a lens of 2 diam. magnification, 3.5X, with a focal distance of 3.5 in. (engraver's lens).

Species Work. Determination of the bacterial flora in water or sewage is not usually worth the labor involved, and in most cases gives no better idea of sanitary condition than tests for *Bact. coli.* For methods and classification, see "General Bacteriology," Jordan, 1925; "Bacteriological Examination of Water Supplies," Savage, Blakiston, 1906; "Elements of Water Bacteriology," Prescott and Winslow, 1924; "Untersuchung des Wassers," Ohlmüller-Spitta, 1910; and other treatises.

Bacteria coli are normal inhabitants of human and animal intestines. Their presence in water usually indicates human or animal pollution. Houston[21] (London, 1902–1903) found from 100 to 1,000,000 *Bact. coli* per g. in feces from 17 normal persons The *Bact. coli* group covers a number of forms or types, but for sanitary purposes is defined as those non-spore-forming bacteria which ferment lactose with gas formation and grow aerobically on standard solid media.

The formation of 10 per cent. or more of gas in a standard fermentation tube within 24 hr. at 37°C. is *presumptive* evidence of the *Bact. coli* group. The appearance of aerobic lactose-splitting colonies on Endo- or eosin-methylene-blue plates made from the tube giving positive presumptive test constitutes a *partially confirmed* test. The *confirmed test* consists in showing that one or more of the aerobic-plate colonies consist of non-spore-forming bacteria which when sown into a fermentation tube will form gas. Results are reported as *presumptive, partially confirmed,* or *confirmed.*

Bact. coli Index. When possible use "Positive," "Negative," or "Pos." or "Neg." or (+) and (−); when no test is made, fill the space with dots (. . . .), not a dash, which might be taken for a minus sign. A zero might be taken for

* Most acid formers are members of the *Bact. coli* group.

either "negative" or "no test." *Bact. coli* index is approximate number of *Bact. coli* per c.c., as determined from qualitative tests upon different quantities of water. For any individual sample, it may be taken as the reciprocal of the smallest volume of water used in the test which gave a positive result. *Bact. coli* index for a single sample is not very accurate. The index becomes more and more precise as the square root of the number of tests becomes larger. *Bact. coli* index may be computed from a series of results as follows: Write down, in order of magnitude of quantities of water examined, the percentages of positive tests, expressed as decimals of 100. Take the differences between these percentages. Multiply each difference by the reciprocal of the quantity corresponding to the larger of the two percentages from which such difference was taken. The sum of these products will be the *Bact. coli* index.

Hale's Method. Dr. F. E. Hale* uses the following method for enumerating *Bact. coli:* Sow six fermentation tubes with portions of sample as follows, 1, 10 c.c.; 2, 5 c.c. and 3, 1 c.c. Estimate number of *Bact. coli* as shown below. The method is useful for surveys of water sources.

Tube No.	Volume tested, c.c.	Results of tests					
1	10	+	+	+	+	+	+
2	5	–	+	+	+	+	+
3	5	–	–	+	+	+	+
4	1	–	–	–	+	+	+
5	1	–	–	–	–	+	+
6	1	–	–	–	–	–	+
Bact. coli in 10 c.c.			5	10	30	70	100

Sewage Streptococci are frequently isolated from polluted sources. Their presence may assist in locating source of pollution.[7b]

Standards of Quality. A sound judgment in regard to the sanitary quality of a water supply should be based on consideration of the facts brought out by a careful sanitary inspection as well as on analytical data. But for administrative purposes, a standard is almost essential.†

The "U. S. Treasury Department Standard." In 1914 the U. S. Public Health Service formulated a standard for the control of water served by common carriers in interstate commerce.[9] This standard was adopted by many municipal and state departments of health. The *standard* has been modified at various times and in 1922[10] three committees were appointed by the Surgeon General of the U. S. Public Health Service to develop a more satisfactory standard. The committee on bacteriological examination has adopted the following standard:

A. Definitions:

1. Index organism: *Bact. coli* group, as determined in accordance with Standard Methods of the A. P. H. A., current edition, by inoculation in lactose broth fermentation tube, transplant to endo- or eosin-methylene-blue agar plate and inoculation in secondary lactose broth fermentation tube.

* Director, Mt. Prospect Laboratory, New York Dept. of Water Supply.

† This standard should serve as a guide rather than to define definite limits, and should not be followed too slavishly.

2. Standard portion of water: 10 c.c.
3. Standard sample of water: 5 standard portions of 10 c.c. each.

B. Limits of permissible density of *Bact. coli* group: not more than 10 per cent. of all the 10-c.c. standard portions examined shall show the presence of organisms of the *Bact. coli* group.

1. When the number of standard samples collected is over 20, not more than 5 per cent. of all the samples shall show three or more positive tests out of five 10-c.c. portions comprised in any single sample.
2. When the number of standard samples collected is less than 20, not more than one sample shall show three or more positive tests out of the five 10-c.c. portions.

British Opinion. In the autumn of 1914 a committee of 13 leading bacteriologists and sanitarians appointed in March, 1914, by the Royal Institute of Public Health of Great Britain, reported:

The committee do not think it is practicable to lay down any fixed standards to govern all cases. Speaking generally, too much stress should not be laid on the number of microbes present in a water, unless the *B. coli* tests yield confirmatory results. A good water should not contain any *B. coli* in 100 c.c., but a water containing *B. coli* in 100 c.c. should not necessarily be objected to without the examination of further samples. Experience has shown that even initially impure waters may be purified at a reasonable cost, so as to yield no *B. coli* in 100 c.c. in the majority (about 75 per cent.) of samples examined. It is much more difficult to suggest a standard by which a water should be condemned. All that the committee feel justified in stating is that the further a water departs from the above standard of purity (no "lactose, + indol, + *B. coli*" in 100 c.c.) the greater is the suspicion attaching to it, unless the local conditions and circumstances are such as to exclude undesirable pollution.

Examples. The following typical analyses illustrate far better than any set rules the meaning of analytical results.

Table 177. Comparative Analyses of Wholesome and Polluted Waters
Parts per Million

Source of sample	Massachusetts mountain spring (unpolluted)	Unpolluted ground water; wells and filter gallery	City supply; surface	Sewage effluent	Polluted well
Odor: Cold, 20° C	00.0	00.0	00.0	00.0	00.0
Odor: Hot, 90° C	00.0	00.0	Vegetable	Musty	2 Vegetable
Turbidity—silica standard	1.	....	....	9.	2.
Color—platinum standard	00.	20.	14.	10.	1.
Oxygen consumed	0.39	....	2.3	2.23	1.02
Nitrogen as: Free ammonia	0.000	0.045	0.013	0.430	0.092
Nitrogen as: Albuminoid ammonia: Total	0.006	0.054	0.097	*5.64	0.074
Nitrogen as: Albuminoid ammonia: Suspended			0.015		
Nitrogen as: Albuminoid ammonia: Dissolved	....	....	0.072	....	....
Nitrogen as: Nitrites	0.000	0.001	0.000	0.214	0.005
Nitrogen as: Nitrates	0.000	0.246	0.031	8.87	36.0
Chlorine	3.00	7.2	2.8	42.2	142.5
Alkalinity	11.0	....	....	....	24.0
Hardness by soap method	14.3	48.0	11.0	....	117.0
Iron, Fe	....	0.46	....	....	....
Total residue on evaporation	32.0	95.7	30.9	296.0.	936.0
Bact. coli in 10 c.c	Negative	Negative	Negative	Positive	Positive
Bact, coli—presumptive test	Negative	Negative	Negative	Positive	Positive

* Total organic nitrogen.

Table 178. Death Rates from Typhoid Fever in Large American Cities Per 100,000 Population[50]

	1924	1923	1922	1921	1916–1920	1911–1915	1906–1910
New England States:							
Fall River	0.0	4.1	3.3	2.4	8.5	13.4	13.5
Hartford	0.0	0.7	2.8	7.2	6.0	15.0	19.0
Worcester	0.5	2.6	3.1	3.2	3.5	5.0	11.8
Springfield	1.3	1.4	1.4	4.4	4.4	17.6	....
Lowell	1.7	2.6	2.6	5.2	5.2	10.2	13.9
Boston	2.1	1.0	1.4	3.1	2.5	9.0	16.0
Providence	2.5	0.8	0.0	2.5	3.8	8.7	21.5
Bridgeport	3.5	1.4	0.6	2.7	4.8	5.0	10.3
Cambridge	4.5	3.6	0.9	10.8	2.5	4.0	9.8
New Bedford	4.5	0.8	0.0	2.3	6.0	15.0	16.1
New Haven	5.1	4.6	5.8	4.7	6.8	8.2	30.8
Middle Atlantic States:							
Trenton	0.8	11.8	15.9	8.9	8.6	22.3	....
Rochester	1.2	1.9	2.5	3.2	2.9	9.6	12.8
Syracuse	1.6	2.2	1.6	3.9	7.7	12.3	15.6
Philadelphia	2.2	1.7	2.7	2.3	4.9	11.2	41.7
Jersey City	2.6	1.6	1.6	3.5	4.5	7.2	12.6
Newark	2.7	2.3	2.7	2.8	3.3	6.8	14.6
Yonkers	2.7	0.9	0.0	2.9	4.8	5.0	10.3
Buffalo	2.8	4.3	3.5	4.2	8.1	15.4	22.8
Scranton	2.8	1.4	1.4	6.5	3.8	9.3	31.5
New York	3.1	2.4	2.2	2.1	3.2	8.0	13.5
Paterson	3.6	2.9	2.1	5.8	4.1	9.1	19.3
Pittsburgh	3.9	3.7	4.6	4.1	7.7	15.9	65.0
Reading	5.4	2.7	9.2	11.7	10.0	31.9	42.0
Camden	6.3	4.0	5.7	6.6	4.9	4.5	....
Albany	12.6	3.4	1.7	4.3	8.0	18.6	17.4
South Atlantic States:							
Richmond	1.1	5.5	4.4	5.6	9.7	15.7	34.0
Baltimore	2.8	4.3	4.0	5.4	11.8	23.7	35.1
Norfolk, Va.	2.9	0.0	6.4	4.1	....	....	....
Washington	4.3	6.0	5.2	6.6	9.5	17.2	36.7
Wilmington	8.3	2.6	5.1	4.4	....	....	....
Atlanta	15.0	17.1	10.9	11.0	11.2	31.4	58.4
East North Central States:							
Akron	1.0	1.4	1.9	5.7	....	....	....
Milwaukee	1.0	1.0	2.5	1.9	6.5	13.6	27.0
Cleveland	1.2	1.7	2.2	3.4	4.0	10.0	15.7
Chicago	1.6	1.9	1.0	1.1	2.4	8.2	15.8
Dayton	2.4	3.6	3.7	5.2	9.3	14.8	22.5
Cincinnati	2.5	3.2	2.9	3.4	3.4	7.8	30.1
Detroit	3.0	4.0	5.0	5.8	8.1	15.4	22.8
Grand Rapids	3.4	1.4	0.6	2.8	9.1	25.5	29.7
Columbus	3.7	4.6	1.1	4.0	7.1	15.8	40.0
Indianapolis	3.9	2.6	5.4	7.3	10.3	20.5	30.4
Toledo	4.4	6.0	3.8	8.6	10.6	31.4	37.5
Youngstown	4.5	7.6	6.2	15.0	....	....	....
East South Central States:							
Louisville	1.9	3.5	8.0	5.5	9.7	19.7	52.7
Birmingham	7.5	7.7	12.5	17.0	31.5	....	....
Nashville	20.4	12.3	16.2	20.4	20.7	40.2	61.2
Memphis	41.2	13.6	8.9	9.0	27.7	42.5	35.3
West North Central States:							
Omaha	1.0	5.4	4.4	4.0	5.7	14.9	40.7
St. Paul	2.0	3.3	2.0	7.1	3.1	9.2	12.8
Minneapolis	2.1	1.0	1.9	1.2	5.0	10.6	32.1
Des Moines	2.8	0.7	1.5	1.4	....	....	....
Kansas City, Mo.	3.6	7.1	4.9	11.0	10.6	16.2	35.6
St. Louis	3.7	4.0	4.2	3.8	6.5	12.1	14.7
Kansas City, Kan.	4.6	0.9	7.0	4.8	9.4	....	....
West South Central States:							
Fort Worth	1.7	4.9	7.8	10.7			
Houston, Tex.	5.6	7.8	7.3	11.7	....	....	....
San Antonio	5.8	9.8	5.6	16.6	23.3	29.5	....
Dallas	8.2	11.6	5.8	12.7	17.2	....	....
New Orleans	10.0	8.7	10.2	9.3	17.5	20.9	35.6
Mountain and Pacific States:							
Oakland	1.3	3.3	3.0	1.3	3.8	8.7	21.5
Spokane	1.9	7.7	5.7	4.7	4.9	17.1	50.3
San Francisco	2.6	3.0	2.2	4.2	4.6	13.6	27.3
Seattle	3.2	2.5	2.8	2.2	2.9	5.7	25.2
Tacoma	3.9	5.9	4.0	3.0	2.9	10.4	19.0
Los Angeles	4.4	3.1	3.7	2.6	3.6	10.7	10.0
Denver	5.1	5.2	5.9	4.5	5.8	12.0	37.5
Portland, Ore.	6.1	2.9	3.3	3.0	4.5	10.8	23.2
Salt Lake City	10.1	4.0	3.2	5.7	9.3	13.2	....

Government Examination of Drinking Water on Railroad Trains. In Bull. 100 of Hygienic Laboratory of U. S. Public Health Service, Treasury Department, Richard H. Creel (November, 1914) pays particular attention to the "bile presumptive test for colon bacillus." He deals especially with anaerobic bacilli, found in water of train coolers, which complicate ordinary so-called coli tests.

The difference between the actual *Bact. coli* percentage in the 1000 samples examined and that which would have resulted had the lactose bile presumptive test been used is as follows, the comparison applying only to tubes containing water in 10 c.c amounts:

Number of samples	Gas	B. coli	Anaerobes	Actual B coli, per cent	Bile presumptive test, B. coli, per cent
1000	421	91	330	9 1	22 1

The margin of error is greater in relatively pure water than in water of moderate pollution.

Estimated according to the different trains, the comparison is as follows:

	Actual *B coli*, per cent confirmed	Percentage of *B coli* according to lactose bile presumptive test	Number of samples
Train 301, Pennsylvania Railroad (sleeping cars).	4 7	25 0	151
Train 84, Seaboard	6 3	17 5	59
Train 305, Pennsylvania Railroad (sleeping cars).	3 4	20 0	125
Train 305, Pennsylvania Railroad (coaches)	5 3	16 0	57
Train 82. Atlantic Coast Line (coaches)	32 0	54 0	50
Train 82, Atlantic Coast Line (sleeping cars)	14 0	17.0	144

It will be noted that there is a wide divergence between the real and the presumptive percentage when the water is fairly pure, but that in moderately polluted water, as that from Train 82, there is less discrepancy. In the latter class of water the approximation of the two figures seems to be due to the inhibiting effect of lactose bile on the *Bact. coli.* From the foregoing results the "presumptive test" for *Bact. coli* does not seem applicable for analyzing water of moderate pollution, as its employment would often result in condemning a water of acceptable standard of purity.

STANDARD METHODS

(Abridged outline of methods based upon Standard Methods for the Examination of Water and Sewage, Am. Public Health Assn., Laboratory Section, 1925.)

Collection of Samples.* Samples should be collected under clean and aseptic conditions and examined as soon as possible. Allowable time between collection and analysis varies; reasonable maximal limits are as follows:

Physical and Chemical Analysis. Ground waters, 72 hr.; fairly pure surface waters, 48 hr.; polluted surface waters, 12 hr.; sewage effluents, 6 hr.; raw sewage, 6 hr. Chemical samples, sterilized by addition of some germicide like chloroform (1 c.c. per L.) may, with impunity, stand a little

* See p. 736.

longer before analysis. Dissolved gases should be determined *in situ*. Care should be taken to collect representative samples.

Bacteriological Examination. Samples kept at less than 10°C., 6 hr.

Microscopical Examination. Ground waters, 72 hr.; fairly pure surface waters, 24 hr.; waters containing fragile organisms, immediately.

Expression of Results. Results of chemical analysis are expressed as parts per million (p.p.m.). This really means milligrams per L. In some older laboratories the results are often expressed either as grains per gal. (U. S. and imp.), or as parts per 100,000.

Table 179. Conversion: Grains per Gal. to Parts per Million

	Grains per U. S. gal.	Grains per Imp. gal.	Parts per 100,000	Parts per 1,000,000
1 gr. per U. S. gal.........	1.000	1.20	1.71	17.1
1 gr. per Imp. gal..........	0.835	1.00	1.43	14.3
1 part per 100,000..........	0.585	0.70	1.00	10.0
1 part per 1,000,000.......	0.058	0.07	0.10	1.0

PHYSICAL AND CHEMICAL EXAMINATION

Turbidity (due to suspended matter) is measured by comparing with a standard suspension of silica (diatomaceous earth) of such a state of fineness that a water which contains 100 mg. per L. silica is one in which the vanishing point of a 1-mm. platinum wire is 100 mm. below surface of water with eye of observer 1.2 m. above wire. Turbidity of this water is fixed at 100. For standards sift dry, precipitated Pear's Fuller's earth through 200-mesh sieve. Suspended 1 g. in 1 L. H_2O for stock solution; turbidity equals 1000. Test a portion diluted to 100 with wire, as above. Turbidity may also be determined by means of U. S. Geological Survey turbidity rod of 1902, provided with the platinum wire mentioned above. Lower rod vertically into the water as far as the wire can be seen; read the turbidity on the graduated rod at surface. Make observations in open air in quiet water, under natural conditions and in the shade.

Turbidity Coefficient. Optical determinations of turbidity (see above) are naturally compared with gravimetric determinations of suspended matter. Equal weights of suspended matter do not necessarily produce same turbidity; *e.g.*, silt or sand produce less than finely divided clay. Therefore, the ratio between silica turbidity determined optically and suspended matter determined gravimetrically is important as an index of the character of the suspended matter. Turbidity Coefficient = Suspended Matter ÷ Silica Turbidity. It varies with different waters, generally increasing with the size of the particles composing the suspended matter.[49]

Color is determined by comparison with standard platinum-cobalt solution. True color is that due to substances in true or in colloidal solution; apparent color includes that produced by suspended matter. Dissolve 1.245 g. K_2PtCl_6 (0.5 g. Pt) and 1 grm. $CoCl_2$, $6H_2O$ (0.25 g. Co) in water with 100 c.c. conc. HCl. Make up to 1 L. This solution has a color value of 500. Dilute and compare sample in tubes by reflected light.

Odor. Observe odor both at 20 and at 90°C. Symbols used to describe odors are as follows:

v—vegetable	m—moldy
a—aromatic	M—musty
g—grassy	d—disagreeable
f—fishy	p—peaty
e—earthy	s—sweetish

Express degree of odor by the following numerals:

Numerical value	Term	Approximate definition
0	None	No odor perceptible.
1	Very faint	An odor that would not be detected ordinarily by the average consumer, but that could be detected in the laboratory by an experienced observer.
2	Faint	An odor that the consumer might detect if his attention were called to it, but that would not attract attention otherwise.
3	Distinct	An odor that would be detected readily and that might cause the water to be regarded with disfavor.
4	Decided	An odor that would force itself upon the attention and that might make the water unpalatable.
5	Very strong	An odor of such intensity that the water would be absolutely unfit to drink. (A term to be used only in extreme cases.)

Oxygen Consumed. $KMnO_4$, 0.4 g. per L.; 1 c.c. = 0.1 mg. O. $(NH_4)\overline{Ox}$,* 0.888 g. per. L. : 1 c.c. = 0.1 mg. O. To 100 c.c. water (if much contaminated dilute smaller portion) in boiling flask add 10 c.c. C. P. H_2SO_4(1:3), heat in water bath 5 min.; add 10 c.c. $KMnO_4$ solution and heat 30 min. in water bath; remove from bath; add 10 c.c. $(NH_4)\overline{Ox}$, then $KMnO_4$ till faint permanent pink. Each c.c. $KMnO_4$ used in excess is equivalent to 1 p.p.m. O.

Nitrogen as Free Ammonia. Standard NH_4Cl solution; 1 c.c. equals 0.01 mg. N (0.0382 g. NH_4Cl in 1 L.). Nessler's reagent: dissolve 50 g. KI in water; add sat. $HgCl_2$ solution until a slight ppt. persists; add 400 c.c. of 9N† clarified KOH; settle and decant. Distil 500 c.c. of sample (with 0.5 g. Na_2CO_3, if sample be acid); collect first three 50-c.c. portions of distillate in Nessler tubes; add 2 c.c. Nessler reagent to each tube; allow to stand 10 min. or more; compare with dilutions of standard NH_4Cl or with permanent standards (see Standard Methods‡). NH_4Cl standard should be made up with ammonia-free water. Where free ammonia is very high it may be determined directly, first clarifying the sample by addition of 10 per cent. solutions of either $CuSO_4$, PbAc,§ or $MgCl_2$, followed by sufficient 50 per cent. KOH to ppt. the metal as hydrate.

Nitrogen as Albuminoid Ammonia. Alkaline permanganate: 8 g. $KMnO_4$ and 400 c.c. of 9N, clarified KOH || to 1 L. Boil off ammonia and make up to volume. Make blank determination and correct for ammonia remaining in

* $\overline{Ox}$ = Oxalic acid or oxalate.
† See foot-note, p. 596.
‡ Standard Methods of Water Analysis, American Public Health Assn.
§ Ac = acetic acid or acetate.
|| NaOH may be used.

reagent. After free ammonia is distilled from water, add 40 c.c. alkaline permanganate and distil until free of ammonia. Four or five 50-c.c. portions should be collected. Nesslerize and compare with standards.

Nitrogen as Nitrites. Sulphanilic acid solution: dissolve 8 g. purest acid in 1 L. 5N* acetic acid (sp. gr. 1.041).

α-Napthylamine solution: dissolve 5 g. in 1 L. 5N acetic acid; filter through washed absorbent cotton.

Standard nitrite solution ($NaNO_2$): dilute strong solution (0.492 g. $NaNO_2$ per L.) 5 c.c. to 1000 parts of water (1 c.c. = 0.0000005 g. nitrogen).

To 100 c.c. of sample in Nessler jars add 1 c.c. of each reagent; mix; allow to stand 15 min. and compare with standards similarly treated.

Nitrogen as Nitrates. Phenolsulphonic acid solution: dissolve 25 g. phenol in 150 c.c. (276 g.) C. P. H_2SO_4; add 75 c.c. fuming H_2SO_4; stir and heat 2 hr. at $\pm 100°C$. Standard KNO_3; 7.2 mg. per L., 1 c.c. = 0.001 mg. N. Prepare by evaporating 10 c.c. of strong solution (0.722 g. KNO_3 per liter) to dryness; treat with reagent and make up to 1 L. Evaporate 20 c.c. or less of the decolorized sample almost to dryness on the water bath; add 1 c.c. of acid to residue; mix well; add water and either KOH or ammonia until alkaline. Compare with standards similarly treated. If chlorides interfere, ppt. with Ag_2SO_4 and filter.

Hardness. Standard soap solution: dissolve 100 g. dry white Castile soap in 1 L. 80 per cent. alcohol and allow to stand several days. From this solution take such quantity that the resulting solution made by diluting stock solution with 70 per cent. alcohol will give a permanent (5-min.) lather when 6.40 c.c. of it are properly added to 20 c.c. of standard $CaCl_2$ (use 75 to 100 c.c. stock solution).

Standard $CaCl_2$: dissolve 0.2 g. pure $CaCO_3$ (calcite) in a little dilute HCl; then add H_2O, and evaporate to dryness several times to expel acid. Make up to 1 L. with H_2O—1 c.c. equivalent to 0.2 mg. $CaCO_3$.

Put 50 c.c. of sample in a 250-c.c. bottle; add standard soap solution slowly until complete lather persists over surface of water 5 min. after shaking. If water be hard, take smaller portions and dilute to 50 c.c. with distilled H_2O. Estimate hardness by means of following table:

Table 180. Hardness: Parts per Million of Calcium Carbonate ($CaCO_3$) for Each Tenth of a Cubic Centimeter of Soap Solution When 50 C.c. of the Sample are Used

C.c. of soap solution	0.0 c.c.	0.1 c.c.	0.2 c.c.	0.3 c.c.	0.4 c.c.	0.5 c.c.	0.6 c.c.	0.7 c.c.	0.8 c.c.	0.9 c.c.
0.0	......	......	......	......	......	......	......	0.0	1.6	3.2
1.0	4.8	6.3	7.9	9.5	11.1	12.7	14.3	15.6	16.9	18.2
2.0	19.5	20.8	22.1	23.4	24.7	26.0	27.3	28.6	29.9	31.2
3.0	32.5	33.8	35.1	36.4	37.7	39.0	40.3	41.6	42.9	44.3
4.0	45.7	47.1	48.6	50.0	51.4	52.9	54.3	55.7	57.1	58.6
5.0	60.0	61.4	62.9	64.3	65.7	67.1	68.6	70.0	71.4	72.9
6.0	74.3	75.7	77.1	78.6	80.0	81.4	82.9	84.3	85.7	87.1
7.0	88.6	90.0	91.4	92.9	94.3	95.7	97.1	98.6	100.0	101.5

*N, as adjective, = normal strength; 5N = five times normal strength; $\frac{N}{50} = \frac{1}{50}$ normal strength.

Table 181. Conversion Table of Hardness

	Parts per million	Clark degrees	French degrees	German degrees
Parts per million	1.0	0.07	0.10	0.056
Clark degrees	14.3	1.00	1.43	0.80
French degrees	10.0	0.70	1.00	0.56
German degrees	17.8	1.24	1.78	1.00

Hardness is expressed according to several arbitrary scales. Other methods for testing hardness are described in connection with water softening (pp. 680, 681).

Chloride. Standard $AgNO_3$ solution, 2.40 g. to 1 L.—1 c.c. equivalent to 0.5 mg. Cl, as chlorides in the water.

Standard NaCl solution, 16.48 g. per L.—1 c.c. equivalent to 1 mg. Cl.

K_2CrO_4 indicator, 50 g. per L. Dissolve salt in a little water; add enough $AgNO_3$ solution to produce precipitate; filter, and make up to 1 L. with distilled water.

$Al_2(OH)_6$ for decolorizing: prepaıe by electrolyzing ammonia-free water using Al electrodes, or by precipitating 125 g. alum, dissolved in 1 L. of water, with ammonia; wash precipitate formed until free from Cl, NH_3, and NO_2.

To 50 c.c. of sample in white porcelain evaporating dish, add standard $AgNO_3$ solution, using 2 c.c. of K_2CrO_4 as indicator, until red tint of Ag_2CrO_4 appears. If chlorine be high, use less than 50 c.c. and dilute to volume; if low, evaporate 250 c.c. or more to volume or add a known quantity of standard NaCl solution and titrate as directed. If color be above 30, add $Al_2(OH)_6$, heat, and filter; if acid, neutralize with soda.

Iron may occur in water both in dissolved and suspended form in ferrous (unoxidized) and ferric (oxidized) condition. Total iron is determined as follows: Standard iron solution: 0.7 g. $Fe(NH_4)_2\ (SO_4)_2,\ 6H_2O$ in 50 c.c. H_2O; add 20 c.c. dilute H_2SO_4, warm slightly, oxidize with $KMnO_4$, and dilute to 1 L.—1 c.c. equivalent to 0.1 mg. Fe. KSCN solution, 2 g. to 100 c.c. Conc. HCl free from iron.

Evaporate 100 c.c. of water to dryness; carefully ignite to destroy excess of organic matter; cool, add 5 c.c. conc. HCl; warm, add 10 c.c. H_2O and warm again. Transfer to 100 c.c. Nessler tube; filter if necessary and add a drop or two of sat. $KMnO_4$ solution. To cool solution add 10 c.c. of KSCN solution and make up to 100 c.c. To a blank tube containing reagent add standard iron solution until color matches that of sample. Compute amount of iron present from number of c.c. of standard solution used.

For determination of iron in different degrees of oxidation, use Standard Methods.

Samples containing small quantities of iron should be concentrated. Samples containing large quantities of iron should be treated with HNO_3, and if necessary, $KMnO_4$; then the iron should be precipitated with ammonia, collected on a filter, redissolved, and determined as above.

Manganese. Standard $KMnO_4$: dissolve 0.288 g. $KMnO_4$ in 1 L. H_2O. 1 c.c. equivalent to 0.1 mg. Mn. HNO_3 (sp. gr. 1.1135) freed from oxides of nitrogen by bubbling through it. Sodium bismuthate, purest obtainable.

Dissolve residue in 10 c.c. dilute HNO_3 plus 1 c.c. conc. H_2SO_4, heat to drive off most of H_2SO_4. Cool: take up with 50 c.c. H_2O and 20 c.c. dil. HNO_3. Add 0.10g. sodium bismuthate, stir, settle; filter through Alundum crucible, wash with 5 per cent. HNO_3, and compare with standards containing known amounts of standard $KMnO_4$. The $KMnO_4$ used for oxygen consumed method contains 0.139 g. Mn per L. Samples containing more than 10 mg. Mn per L. should be tested by Knorres' persulfate method (see Standard Methods).

Residue on Evaporation. Weigh platinum dish; evaporate to dryness 100 c.c. or more of sample; dry at 104°C. and weigh. Gain in weight is total residue in volume of water tested. Same procedure with filtered samples determines dissolved residue. Difference in weights of residues before and after ignition gives "loss on ignition." Final weight after ignition is that of "fixed residue."

Acidity. Titrate 100 c.c. of sample with $\frac{N^*}{50}$ Na_2CO_3, using erythrosine or methyl red. Methyl orange is unsuitable in presence of free aluminum sulphate. Express results in terms of $CaCO_3$.

Carbon Dioxide. Standard $\frac{N^*}{44}$ NaOH. 1 c.c. = 1 mg. CO_2. Dissolve 0.91 g. NaOH in 1 L. CO_2 free water. Preserve in resistant glass bottles. Titrate rapidly 100 c.c. of sample, using phenolphthalein as an indicator. Use a Nessler tube and stir gently. Each c.c. $\frac{N}{44}$ NaOH $\times$ 10 = 1 p.p.m. free CO_2. When water is acid to phenolphthalein, then:

Bicarbonate = 1.22 times the alkalinity.
Carbonic acid as bicarbonate = 0.88 times the alkalinity.
Half-bound carbonic acid = 0.44 times the alkalinity.
Carbonate = 1.22 times the alkalinity with phenolphthalein.

Table 182. Relation between Alkalinity by Phenolphthalein and that by Erythrosine in Presence of Bicarbonates, Carbonates and Hydrates

	Bicarbonates	Carbonates	Hydrates
$P=0$	E	0	0
$P<\frac{1}{2}E$	$E-2P$	2P	0
$P=\frac{1}{2}E$	0	2P	0
$P>\frac{1}{2}E$	0	$2(E-P)$	$2P-E$
$P=E$	0	0	E

E = Erythrosine alkalinity. P = Phenolphthalein alkalinity.

Carbonic acid (CO_2) as bicarbonate = { 0.88 times the alkalinity with erythrosine or methyl red.

Bound carbonic acid, as CO_2 = 0.44 times the alkalinity with erythrosine or methyl red.

$$\text{Half-bound carbonic acid} = \frac{\text{bicarbonate carbonic acid}}{2}$$

Bound carbonic acid, as CO_2 = $\frac{1}{2}$ (carbonic acid as carbonate)

*See footnote, p. 596.

Dissolved Oxygen—Winkler's method,[52] Rideal-Stewart modification.
$MnSO_4$, 48 g. to 100 c.c. water.
KI + NaOH, 50 g. NaOH and 15 g. KI to 100 c.c. water.
Conc. H_2SO_4, sp. gr. 1.84.
Thiosulfate solution, 6.2 g. $Na_2S_2O_3$ to 1 L. water. $\left(\frac{25\,N}{1000}\right)$ – 1 c.c. solution = 0.2 mg. O (or 0.1395 c.c. at 0°C. and 760 mm. pressure).
$KMnO_4$, 6.32 g. to 1 L.
$K_2\overline{OX}$, 10 g. to 1 L.
Starch – 1 per cent. solution sterilized.

Table 183. Solubility of Oxygen in Fresh Water and in Sea Water of Stated Degrees of Salinity at Various Temperatures When Exposed to an Atmosphere Containing 20.9 Per Cent. of Oxygen and Under a Pressure of 760 Mm.*

Calculated by G. C. Whipple and M. C. Whipple from Measurements of C. J. Fox

Temperature, centigrade	Chlorine in sea water (parts per million)					Difference per 100 parts of chlorine per million
	0	5000	10,000	15,000	20,000	
			Milligrams per liter			
0	14.62	13.79	12.97	12.14	11.32	0.0165
1	14.23	13.41	12.61	11.82	11.03	0.0160
2	13.84	13.05	12.28	11.52	10.76	0.0154
3	13.48	12.72	11.98	11.24	10.50	0.0149
4	13.13	12.41	11.69	10.97	10.25	0.0144
5	12.80	12.09	11.39	10.70	10.01	0.0140
6	12.48	11.79	11.12	10.45	9.78	0.0135
7	12.17	11.51	10.85	10.21	9.57	0.0130
8	11.87	11.24	10.61	9.98	9.36	0.0125
9	11.59	10.97	10.36	9.76	9.17	0.0121
10	11.33	10.73	10.13	9.55	8.98	0.0118
11	11.08	10.49	9.92	9.35	8.80	0.0114
12	10.83	10.28	9.72	9.17	8.62	0.0110
13	10.60	10.05	9.52	8.98	8.46	0.0107
14	10.37	9.85	9.32	8.80	8.30	0.0104
15	10.15	9.65	9.14	8.63	8.14	0.0100
16	9.95	9.46	8.96	8.47	7.99	0.0098
17	9.74	9.26	8.78	8.30	7.84	0.0095
18	9.54	9.07	8.62	8.15	7.70	0.0092
19	9.35	8.89	8.45	8.00	7.56	0.0089
20	9.17	8.73	8.30	7.86	7.42	0.0088
21	8.99	8.57	8.14	7.71	7.28	0.0086
22	8.83	8.42	7.99	7.57	7.14	0.0085
23	8.68	8.27	7.85	7.43	7.00	0.0083
24	8.53	8.12	7.71	7.30	6.87	0.0083
25	8.38	7.96	7.56	7.15	6.74	0.0082
26	8.22	7.81	7.42	7.02	6.61	0.0080
27	8.07	7.67	7.28	6.88	6.49	0.0079
28	7.92	7.53	7.14	6.75	6.37	0.0078
29	7.77	7.39	7.00	6.62	6.25	0.0076
30	7.63	7.25	6.86	6.49	6.13	0.0075

* Under any other barometric pressure, B, the solubility can be obtained from the corresponding value in the table by the formula:

$$S' = S\frac{B}{760} = S\frac{B'}{29.92}$$

S' = solubility at B mm. = B' in.
S = solubility at 760 mm. = 29.92 in.

Collect sample in 250 c.c. calibrated, stoppered bottles without exposure to or absorption of atmospheric air. Take temperature. Add by pipettes reaching below surface, 0.7 c.c. acid and 0.1 c.c. permanganate sol. Destroy excess with $K_2\overline{OX}$ add 1 c.c. of $MnSO_4$ and 3 c.c. KI + NaOH solution. Stopper bottle, shake, and allow ppt. to settle. Remove stopper; add 1 c.c. H_2SO_4; replace stopper and dissolve ppt. by shaking. Remove contents to flask and titrate with thiosulfate using starch as an indicator.

$$\text{Oxygen, p.p.m.} = \frac{200N}{V}$$

$$\text{Oxygen, c.c. per L.} = \frac{139.5N}{V}$$

$$\text{Oxygen, per cent. saturation} = \frac{20,000N}{VO}$$

where N = c.c. thiosulphate solution.

V = capacity of bottle in c.c. less volume of reagents (4 to 5 c.c.).

O = p.p.m. of O in water saturated at same temperature and pressure (Table 183).

Free chlorine in waters, treated in excess, may be detected by KI and starch solution in the sample acidified with H_2SO_4. For quantitative estimation, ortho-tolidine method is used (Ellms and Hauser, *J. Ind. Eng. Chem.*, Vol. 5, No. 11, 1913).

Ortho-tolidine solution: Dissolve 1 g.* in 1 L. water containing 100 c.c. conc. HCl. Add 1 c.c. of test solution to 100 c.c. of water and compare color with that produced in standard solutions containing known amounts of chlorine. Standardize chlorine solution against $\frac{N}{100}$ thio-sulfate solution. Use redistilled water for diluting standards and compare rapidly. Permanent standards of $CuSO_4 + K_2Cr_2O_7$ may be used. Test will detect 0.005 p.p.m. free Cl

Standard Bacteriological Media. Standard media may be purchased in dry form ready for dilution and sterilization. The constituents for preparing the principal media are as follows:

Nutrient Broth. Three grams beef extract and 5 g. peptone to 1 L.

Sugar Broths. Nutrient broth + 0.5 per cent. of required carbohydrate (lactose, dextrose etc.).

Nutrient Gelatine. Three grams beef extract, 5 g. peptone and 100 g. gelatin dried at 105°C. before weighing.

Nutrient Agar. Like gelatin with 12 g. agar in place of 100 g. gelatin.

Litmus Solution. Two per cent. solution of reagent litmus.

Azo-litmin Solution. One per cent. solution of Kahlbaum's azo-litmin.

Azo-litmin Agar. Ten grams peptone, 2 g. K_2HPO_4, and 15 g. agar to 1 L. Store in flasks or bottles. Sterilize just before using, melt, and add to each 100 c.c. 5 c.c. of 20 per cent. lactose solution, 2 c.c. of 0.5 per cent. methelene blue solution. Pour medium into Petri dishes for use.

Endo Medium. Ten grams peptone, 30 g. of agar (dried) to 1 L. Store in flasks. Sterilize. Prepare 10 per cent. solution of basic fuchsin in 95 per cent. alcohol; settle, decant, and filter it. When ready, melt 100 c.c. of stock agar.

* Pure salt may be obtained from Eastman Kodak Co.

Dissolve 1 g.* C.P. lactose in 15 c.c. of water and 0.25 Na_2SO_3 in 10 c.c. of water. Add 0.5 c.c. of fuchsin solution to the Na_2SO_3 solution; add the mixture to the lactose solution and then add the combination to the melted agar. Pour plates and harden.

Sterilization of media is complete after 10 min. in an autoclave under 15 lb. pressure (120°C.).

CHARACTERISTICS OF CLASSES OF WATER

Meteoric Water. All water derived directly from the atmosphere is called meteoric water, whether in the form of dew, rain, snow, sleet, or hail. Water evaporated from fresh-water lakes and condensed on high mountains is exceptionally pure, but rain and snow collected in towns is more or less polluted with the washings of air and roofs, especially during the beginnings of storms. Examples of rain and snow water are given in Table 207 (p. 622) (samples 69 to 75).

Chlorine in Rain Water. Along the coast, rain water contains small amounts of sodium chloride borne from the sea, the amount varying with the distance from the sea and the direction of the air currents. See chlorine map, Fig. 247, p. 586.

Gases in Rain Water. For solubilities of atmospheric gases in water see p. 583, and any chemists handbook, *e.g.*, "The Chemists Year Book," "Van Nostrand's Chemical Annual," "Die Chemiker Kalender."

Rain water contains traces of the omnipresent ammonia and nitrates, and occasionally traces of nitrites. The amount of nitrogen as free ammonia varies between 0.2 and 5.0 p.p.m., and as nitrate from 0.2 to 2 p.p.m. (Table 207, p. 622). Rain water contains ozone (O_3). This is not a purifying agent, *per se*. The so-called ozone process of water purification (see p. 665) depends upon the production of traces of hydrogen peroxide (H_2O_2).[11] Angus Smith found 35 p.p.m. of sulphuric acid in rain water in Liverpool, 50 parts in Manchester, 430 parts in Newcastle, and 20 parts in London. In the vicinity of smelters and chemical works, rain water may contain hydrochloric, nitric, and other acids, nitrous oxide, carbon monoxide, and oxides of zinc, lead, copper, and other metals.

Ground Water.* The meteoric water which sinks into the ground and flows seaward along more or less impenetrable strata is called ground water—subdivided for convenience into spring, shallow-well, deep-well, and artesian water.

Appearance. Meteoric water falling upon the earth and passing through thick enough layers of proper soil loses its suspended matter and becomes clear. An exception is found where fine particles of clay are borne with the water through the relatively large pores of water-bearing soil or the fissures and seams in the rock. Waters which become turbid directly after rains are usually polluted from surface sources. Some waters which are clear when drawn become *turbid* on standing, due to the precipitation of iron, manganese, calcium, and magnesium carbonates, etc., generally following the escape of the gases which hold these substances in solution. Ground waters are usually *colorless*, but not always so. The deep well waters of the Mississippi delta

* See also Chap. 4.

often have a color of 1400 and waters from near peat deposits are also colored. The color is due to humus bodies of obscure composition (humic acid and ulmic acid). These are frequently associated with iron and manganese (iron humate, etc.).

Odor. Most ground waters have little or no odor. Some, however, are impregnated with hydrogen sulphide, methane or sulphur dioxide and possess the characteristic odors of these gases. Polluted wells may have a musty or disagreeable odor, while ground waters from marshy sources often give off the characteristic odor of peat, especially when heated.

Tastes of ground waters are usually imparted by dissolved gases and minerals. The agreeable taste of carbonic acid and the disagreeable taste of hydrogen sulphide are well known; waters containing iron or manganese are characterized by a styptic or "inky" taste; the hardness of water is often apparent to the taste, especially when large amounts of magnesium are present. Some waters contain so much sodium chloride that it can be tasted. The minimum amount of sodium chloride which can be so detected ranges, according to various observers, from 200 to 300 p.p.m. Few observers can detect 200 p.p.m. or less of sodium chloride, even when dissolved in distilled water.

*Temperature.** One valuable property of ground waters is the small annual range in temperature. The temperature of shallow-well waters is rarely over 20°C., and rarely lower than 6°C. Waters from greater depths are practically constant in temperature. E. A. Martel[15] states: "A constancy of temperature exists only in the continuous flows from sand and similar material or artesian supplies from great depths, slowly and regularly maintained." Ground waters are cooler than surface waters in summer and are, therefore, more agreeable to drink; in winter, ground waters are warmer than surface waters and are less liable to cause freezing of services.

Organic matter in the soil and that passing through it with the ground water are of two main kinds, carbonaceous and nitrogenous. Theoretically, these may become oxidized to CO_2, CH_4, and H_2. These end products are rarely reached in nature as the result of decomposition of organic matter in the soil. Carbonaceous matter comprises the cellulose, lignin, and chemically similar bodies; the nitrogenous matter comprises the albumins, the waste of animal life, and proteids. Carbonaceous matter may be oxidized to carbon dioxide, and the nitrogenous matter may be oxidized to nitrates. Rideal[12] divides nitrogenous decomposition into three periods: first, the formation of polypeptides from proteids, a period of which sewage is a typical example; second, the production of amino-acids; last, the conversion into ammonium compounds, which are oxidized into nitrites and nitrates. Organic matter and the bacteria which decompose it, are most abundant in the surface layer of the soil, where also there is the largest amount of available oxygen.

The circulation of nitrogen from the organic to the mineral state is known as nitrification, and is brought about by the nitrifying bacteria (discovered by Winogradsky). These bacteria change ammoniacal nitrogen to nitrites and nitrates. Nitrates are readily taken up by green plants through the constructive power of chlorophyll. When the living matter again breaks

* Data regarding temperatures for industrial use are given in *Water Supply & Irrigation Paper* 520, 1925.

down, free ammonia and nitrates and nitrites are formed as before. This cycle is illustrated in Fig. 248 devised by Fowler[13] (1911), which illustrates the transformations of nitrogen in nature in a never-ending cycle, through the vitalized and mineralized stages in turn. Certain bacteria, called denitrifying, reduce nitrates with the production of nitrites. Some bacteria, in turn, reduce nitrites to ammonia and ammonia to nitrogen, thus reversing the ordinary course of the nitrogen cycle.

Dissolved Gases. Carbon dioxide, which results from decomposition of organic matter, is the gas of first importance in ground waters, which contain little or no oxygen. Meteoric waters contain little carbon dioxide but are nearly saturated with oxygen. As meteoric water passes into the soil, its oxygen is used to decompose organic matter contained in the soil and water. Fre-

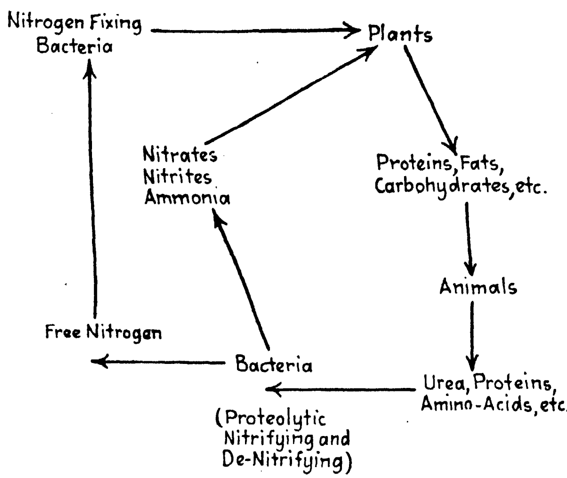

FIG. 248.—The nitrogen cycle.[13]

quently deep wells contain extraordinary amounts of nitrogen in the form of free ammonia, particularly where there are fossil, animal, or vegetable deposits in the water-bearing strata. Bartow[14] reports several cases illustrating this phenomenon. Such waters from sources of unquestioned purity, may contain even more than 2.0 p.p.m. of nitrogen as free ammonia.

Effect of Carbon Dioxide. Production and absorption of CO_2 greatly increase the solvent action of water. Lime, magnesia, iron, manganese, and other elements are rendered soluble, while the silicates and other minerals are decomposed, setting free the soluble elements composing them. Ground waters, therefore, contain more substances in solution but less in suspension than do surface waters. Humic or other organic acids may exert a similar effect on the soil. If dissolved oxygen be absent from the ground water, sulfates may be reduced to sulfides and hydrogen sulfide set free. Similarly, nitrogenous organic matter is reduced, with the production of ammonia and nitrogen.

Bacteria in Ground Water.* Numbers of bacteria in soil and in ground water decrease rapidly with depth. Kabrehl[16] found several million bacteria

* See Public Health Reports, *E. N. R.*, Sept. 13, 1923, p. 425.

per g. in surface samples of woodland soil, a few thousands 0.5 m. below the surface, and usually only hundreds per c.c. in samples collected from depths greater than 1 m. Many organisms in ground water grow slowly and therefore do not appear after short periods of incubation, thus giving rise to the erroneous belief that ground waters from considerable depths are invariably bacteria free. S. C. Prescott[7a] examined 147 shallow farm yard wells and found that 124 contained no *Bact. coli* and were therefore probably free from fecal pollution. These samples averaged 190 bacteria per c.c., while the 23 samples which gave positive tests for *Bact. coli* averaged 570 per c.c. Distribution of the two series of samples according to the number of bacteria present is indicated in Table 184.

Table 184. Bacteria in Shallow Farmyard Wells

Percentage of Samples in Each Group[7a]

Bacteria per c.c.	0	1–10	11–20	21–50	51–100	101–500	501–1000	1001–2000	2001–3000
Series I. Bact. coli absent..	3	16	14	16	11	31	5	4	....
Series II. Bact. coli present.	..	....	5	.....	10	57	10	14	5

The following table gives a summary showing the relative purity of water collected from wells of various depths by the Illinois State Water Survey[17] during 1910.

Table 185. Relative Purity of Illinois Well Waters from Various Depths

	Depth, ft.					
	Under 25	25 to 50	51 to 100	Over 100	Unknown	Total
Number Examined.	148	201	90	205	67	711
Number Condemned	118	137	46	43	35	379
Per cent. Condemned	79+	65+	51+	20+	52+	53+

Kellerman and Whittaker[18] report similar results for *shallow* wells used as farm water supplies. Egger[19] examined 60 wells in Mainz and found that 17 of them contained over 200 bacteria per c.c. Maschek (1887) found 36 wells out of 48 examined in Leitmeritz which had a bacterial content of over 500 per c.c. Fischer[20] reported 120 wells in Kiel which gave over 500 bacteria per c.c. and only 51 with less.

Deep well waters contain very few bacteria. Houston[21] found that the number of bacteria in the waters from a series of deep wells of high quality at Tunbridge Wells, England, varied from 1 to 36 per c.c. Prescott[7a] found in the water of 15 driven wells in the vicinity of Boston, examined in 1903, an average of 18 colonies per c.c. at the end of 48-hr. incubation. Prescott and Winslow[7a] found the following numbers of bacteria in certain wells and springs from various sources:

Table 186. Bacteria in Deep Well and Spring Waters

Town	Bacteria per c.c.	Town	Bacteria per c.c.
Worcester, Mass...........	10	Saranac Lake, N. Y..........	11
Waltham, Mass...........	3	Ellenville, N. Y..............	0
Newport, R. I...........	7	Hyde Park, Mass............	12

Tanner and Bartow[22] found the variations in the bacteriological content of water from some deep wells as shown in the following table.

Table 187. Bacterial Content of Waters from Deep Wells in Illinois[22]

Location	Depth (feet)	Bacteria per c c	
		Agar 37°	Gelatin 20°
Joliet	282	9	55
Champaign	400	0	0
Champaign	160	1	1
Champaign	160	28	30
Champaign	120		6
Danvers	211	4	10
Waukegan	895	0	20
Lexington	113	4	3
Fairbury	2000	1	9
Dwight	126	0	2
Dwight	126	0	6
Red Bird	270	4	10

Bacteria Coli in Ground Waters. *Bact. coli* are found in polluted ground waters. *Bact. coli* may occasionally be found in ground waters of good quality, provided large enough volumes be tested. The bacteria abound at the ground surface and a very few may penetrate to great depths. Therefore it is the number found in a ground water, as in a surface water, rather than their mere presence, which is of sanitary importance.

Massachusetts State Board of Health (Report 1901) examined samples from 99 springs and found *Bact. coli* in six. All were liable to pollution. The same investigation showed that bottled spring waters were subject to contamination during bottling, *Bact. coli* being found in seven samples where none were found in the corresponding samples from the springs. Clark and Gage[23] found *Bact. coli* in five out of 170 samples from tube and shallow wells of good construction. Houston (*Ref.* 21) compares the more or less polluted shallow wells of Chichester with the excellent deep ground waters from Tunbridge Wells. Table 188[7e] shows the value of 1-c.c. samples in differentiating between good and bad waters. Fromme[34] shows relation between numbers of bacteria and *Bact. coli* as found by examining 120 samples from wells near Hamburg (Table 189). Certain exceptional waters from the vicinity of

Table 188. Distribution of Bact. Coli in Good and Bad Well Waters
Houston, 1903[7e]
Percentage of Positive Tests; 1 C c Portions

Quantity of water	Chichester, shallow wells	Tunbridge Wells, deep wells
100 c c.	90	25
10 c.c.	80	6
1 c.c.	45	0
0 1 c c.	20	0

Table 189. Relation between Total Numbers of Bacteria and Bact. Coli

Colony count	Number of samples	Per cent. positive; B. coli tests in 10 c.c.
Over 200	35	40.0
50–200	19	15.8
Under 50	66	3 0

peaty deposits, such as exist on Cape Cod, Long Island, and elsewhere, contain bacteria showing most of the *Bact. coli* reactions. Such waters should not be judged by the presumptive test for *Bact. coli* alone.

Mineral Content of Ground Water.[49] According to Tiemann and Gärtner,[35] the substances dissolved in normal ground waters of North Germany lie, as a rule, within the following limits, which limits exclude strongly impregnated mineral waters:

Substance	Parts per Million
Total residue on evaporation	Less than 500
Calcium and magnesium (Ca + Mg)	Less than 150
Chlorine (Cl)	Less than 30
Sulfates (SO_4)	Less than 120
Nitrogen as nitrates	Less than 5
Nitrogen as nitrites	Less than 0.01
Nitrogen as free ammonia	Less than 1
Oxygen consumed (O)	Less than 2.5

Most of the acids and bases present in natural waters are in the combined state. Reichardt[49] states the composition of ground waters from various geological formations in a table which has been frequently republished.

For a bibliographical review of all literature concerning the mineral analyses of American ground waters, see *Reports of U. S. Geological Survey, Water Supply Papers* 120, 163, 427, 489.

Sources of Pollution. Wells and springs may become polluted by surface drainage (see p. 629), entering either through a fissure or through poorly constructed casings, curbs, or cover (see p. 630). Infiltration galleries in coarse material are liable to contamination during flood times.

Surface waters from streams, rivers, ponds, and lakes, the most generally available sources, are especially liable to pollution, and for this reason are rarely suitable for domestic use without purification, either by storage (natural or artificial), chemical treatment, filtration, or disinfection. Analyses of various surface waters given in Table 207 (samples 1 to 68) (p. 620) show better than a description their varying characters. They differ greatly in suspended matter, including microscopic organisms and bacteria. They have temperatures largely dependent upon climate and season and a varying gaseous content which, in the case of pure waters, is characterized by low CO_2 and high O and N. Certain waters may be supersaturated with oxygen produced by active growth of algæ. Other characteristics of surface water are the generally higher amounts of nitrogenous and carbonaceous organic matter, as shown by higher "loss on ignition," "albuminoid ammonia," color, and "oxygen consumed." Some ground waters which contain humus matter exhibit these characteristics. Surface waters contain less dissolved mineral matter (Si, Ca, Mg, Na, and K) and practically no dissolved iron or manganese except that which is in combination with the organic coloring matter.

Dissolved Gases in Typical Waters. Gases expelled by boiling five typical waters were determined by Sir E. Frankland, with results given in Table 190. They included rain water and water from a deep well in the chalk deposit, as well as from three surface sources. The high CO_2 content in the Thames and

[49] See "Grundlagen zur Beurteilung des Trinkwassers," E. Reichardt (Halle, 1880).

chalk well waters is due to oxidation of organic matter. The values given include not only the free CO_2, but that in the form of bicarbonates, which also is driven off by boiling. Thresh[36] found 99.16 per cent. of N, and 0.84 per cent. of CO_2 in gases evolved from Buxton thermal spring, in England.

Table 190. Gases Expelled by Boiling Typical Waters, c.c. per L.
(Frankland)[48]

Kind of gas	Rain water	Cumberland mountain water	Loch Katrine water	Thames water	Deep chalk well water
Nitrogen	13.08	14.24	17.31	13.25	19.44
Oxygen	6.37	7.26	7.04	5.88	0.28
Carbonic acid	1.28	2.81	1.13	40.21	55.20
	20.73	24.31	25.48	59.34	74.92

Organic Matter in Surface Water. Suspended organic matter includes that organized as microscopic plants and animals, the former including the bacteria. These are true plants although some of them possess the power of locomotion. It also includes wastes of plant and animal life and organic matter discharged with sewage from factories and towns. Growth of organisms in surface water is dependent upon food and light. Nitrates and carbon dioxide form the ideal plant food and give rise to abundant growths of algæ. Bacteria which feed upon or peptonize the suspended matter, and bacteria which mineralize the dissolved organic matter, with the algæ, furnish food for growths of Protozoa; these, in turn, form part of the food for growths of Hydræ and Rotifera, all of which are consumed by fish. Oysters and clams also feed upon diatoms and other microscopic organic matter, living and dead; and the abundance of the higher plant life supported indirectly by the organic matter contained in the water in which it grows is well known. Often the only apparent result of increased organic pollution of a body of water is a corresponding increase in fish, oysters, clams, etc., for example, Illinois River[37] and Genesee River.[38]

Fecal waste from the higher animal forms decomposes into substances which, in the presence of oxygen, pass through the nitrogen cycle shown in Fig. 248 (p. 603). Some organic matter present in a water is removed in the form of the adult insects whose larvæ live in the water, for example, Chironomas. Solid masses of organic sludge from the bottoms of streams become gradually liquefied by anaerobic bacteria. These sludge masses also serve as food for a variety of worms and larvæ, but only in the presence of oxygen. Liquefied sludge forms part of the food for bacteria and other organisms. There exists, therefore, in many streams containing undecomposed organic matter, a continuous cycle of changes, resulting ultimately in biological equilibrium and, if no new food material be forthcoming, in rapid decadence of organic life and mineralization of the dead organic matter. In other words, growth of organisms, including bacteria, plays the most important rôle in self-purification of streams.

Much organic matter in surface waters is in the colloidal state (see p. 651). This matter slowly coagulates and subsides. For example, the coloring

matter in the highly colored Dismal Swamp water precipitates on standing, although the water apparently contains no suspended matter when first drawn.

For names and descriptions of genera of organisms, as well as their frequency of occurrence, their relation to the chemical constituents of the water and to sizes and depths of reservoirs, see Whipple's "Microscopy of Drinking Water" (Wiley, 1914); also Ward and Whipple, "Fresh Water Biology" (Wiley, 1918). Frequency of occurrence of the various groups in Massachusetts waters is summarized numerically in Table 191. Ten genera

Table 191. Frequency of Occurrence of Groups of Organisms in Massachusetts Surface Waters[40d]

Classification	Number of genera				
	Commonly found in large numbers	Occasionally found in large numbers	Commonly found in small numbers	Occasionally observed	Total
Diatomaceæ.	5	4	4	22	35
Chlorophyceæ..	3	8	14	21	46
Cyanophyceæ..	4	3	1	8	16
Fungi and Schizomycetes	1	3	3	5	12
Protozoa	5	5	11	24	45
Rotifera	0	0	5	12	17
Crustacea	0	0	3	4	7
Miscellaneous	0	0	0	10	10
Total ..	18	23	41	106	188

are especially troublesome in Massachusetts, namely Asterionella, Anabæna, Clathrocystis, Cœlosphærium, Aphanizomenon, Dinobryon, Peridinium, Synura, Uroglena, and Glenodinium.

Whipple[40f] made a statistical study of 66 lakes and reservoirs in New England* and classified them according to their *index of frequency*, calculated as follows:

It was assumed that when organisms were less than 500 per c.c. they would cause no trouble; between 500 and 1000 per c.c., little trouble; between 1000 and 2000, noticeable trouble; between 2000 and 3000, decided trouble, and that above 3000, trouble would be serious. From analyses the per cents of the time when organisms were present within these limits were ascertained. These were then weighted as follows and added together: For numbers between 500 and 1000, one-half the per cent.; for numbers between 1000 and 2000, the per cent. as computed; for numbers between 2000 and 3000, twice the per cent.; and for numbers above 3000, three times the per cent. The above was based on organisms of all kinds disregarding genera. An index of 50 would mean that organisms were noticeable half the time, or that if they were present for less than half the time they were more troublesome during the time when they were present.

The maximum index of frequency possible by this method of computation is 300. Natural waters rarely have an index of more than 100. Best stored waters have an index less than 10 (see Table 192).

In Northeastern New Jersey[51] the numbers of organisms in reservoirs bear close relation to their storage ratios, as shown in Table 193.

* For other parts of the U. S. no reports of regular examinations are published.

Table 192. Lakes Classified According to Index of Frequency of Organisms

Whipple[40b]

	Group I	Group II	Group III
Numbers of lakes and reservoirs in the group....	28	18	20
Limits of frequency index..........	0–25	25–50	50–100
Average index of frequency.......	12	39	78
Organisms per c.c., mean yearly average.......	362	776	1410
Organisms per c.c., minimum yearly average.	54	441	984
Organisms per c.c., maximum yearly average..	1413	2800	3090
Organisms per c.c., mean average for 4 summer months..........	414	1023	1965
Organisms per c.c., minimum average for 4 summer months	66	227	985
Organisms per c.c., maximum average for 4 summer months ...	1058	4588	7659

Group I Organisms often as high as 1000 per c.c. Group II Organisms only occasionally as high as 1000 per c.c. Group III Organisms ordinarily between 100 and 500 per c.c.

Table 193. Showing Relation between Storage Ratio and Microörganisms of Various Reservoirs in the Wanaque and Pequannock Catchment Areas, Respectively

(Weston)[51]

Body of water	Catchment area, square miles	Capacity, mg	Yearly inflow, mg	Storage ratio	Micro-organisms standard units, c.c.
Sterling Lake	5.1	3,586	2,360	1.52	93
Greenwood Lake	27.1	14,079	11,354	1.24	186
Canistear Reservoir	5.6	2,407	2,625	0.92	366
Clinton Reservoir	10.5	3,518	4,935	0.71	558
Pequannock, Total	63.7	11,808	29,890	0.39	518
Oak Ridge Reservoir	27.3	3,982	12,800	0.31	569
Echo Lake	4.6	612	2,155	0.28	579
Forge Pond	14.7	7.5	7,090	0.11	652

Table 194. Algæ in Connecticut Water Supplies[40a]

City	Lake or reservoir	Index of frequency
Bridgeport ...	Island Brook Supply	83
New Haven..............	Dawson Lake	79
Meriden	Merimer Lake	74
New Britain..	Shuttle Meadow Lake	55
Middletown	Laurel Brook Reservoir	45
Bridgeport.............	Pequonnock River Supply	40
Hartford.................	Reservoir No. 6	29
Norwich................	Fairview Reservoir	22
New Haven.	Whitney Lake	21
Hartford...............	Reservoir No. 1	18
Bridgeport..............	Mill River Supply	18
Hartford...............	Reservoir No. 3	12
New Haven..............	Wintergreen Lake	12
New Haven..............	Saltonstall Lake	0
Hartford.................	Reservoir No. 2	0
Hartford..................	Reservoir No. 5	0
New London.........	Lake Konomoc	0

Bacteria in Surface Waters. Most rivers in inhabited regions contain several hundreds or thousands of bacteria per c.c., dependent upon the degree of pollution. Furthermore, these numbers fluctuate rapidly, especially during storms and floods. Table 195 illustrates these seasonable variations.

Table 195. Seasonal Variations in Bacterial Content of River Waters. Bacteria per c.c. Monthly Average[7f]

River	Year	Jan.	Feb.	Mar.	April	May	June
Thames*	1905–6	2,075	1,678	1,161	277	1,064	382
Lea*	1905–6	5,192	3,083	1,308	471	1,350	598
New*	1905–6	1,455	1,304	291	149	352	198
Mississippi†	1900–01	972	2,871	1,795	3,597	2,152	2,007
Potomac‡	1906–7	4,400	1,000	11,500	3,700	750	2,300
Merrimac‖	1905	14,200	14,800	10,300	3,600	1,900	9,600
Susquehanna§	1906	9,510	21,228	31,326	39,905	6,187	2,903

River	Year	July	Aug.	Sept.	Oct.	Nov.	Dec.
Thames*	1905–6	952	——	——	——	1,633	740
Lea*	1905–6	1,190	——	——	——	3,946	2,050
New*	1905–6	450	——	——	——	718	621
Mississippi†	1900–01	1,832	805	——	——	——	2,021
Potomac‡	1906–7	2,700	3,000	6,200	2,300	1,800	6,900
Merrimac‖	1905	3,900	19,500	13,500	39,800	8,700	——
Susquehanna§	1906	685	1,637	836	7,575	26,224	37,525

* Houston, 1906*a*, 1906*b*.
† New Orleans, 1903.
‡ Figures obtained through courtesy of F. F. Longley.
‖ Massachusetts, 1906.
§ Harrisburg, 1907.

In warm weather and during periods of low flow, putrefaction is likely to occur in river water, greatly changing its bacterial content. This condition is accompanied by loss of oxygen, sometimes to its entire disappearance and the destruction of fish life, and may indirectly cause an increase in bacteria. Temperature itself has but slight influence upon the numbers of bacteria found, although growths of antagonistic microscopic organisms are most frequent during the warmer months.

Discharge of sewage affects the number of bacteria markedly. Bacteria in the Niagara River vary between 10,000 and 300,000 per c.c., and average 25,000 per c.c., dependent upon the degree of sewage contamination.[41] According to Miquel,[42] there are in the Seine River above Paris 300 bacteria per c.c. and below Paris (Clichy) 200,000 per c.c. Koch[43] found in the Spree 82,000 bacteria per c.c. above Koepenick and 10,000,000 per c.c. at Charlottenburg some miles below. Schlatte[44] found in the Limmat 1000 to 2000 bacteria per c.c. before the addition of the sewage of Wipkingen, and 5000 thereafter. The Spree contains 2,500,000 more bacteria per c.c. below Berlin than above. Various disinfecting wastes, like those containing

acid, copper, or chlorine, cause reductions in bacterial contents and in numbers of *Bact. coli* when discharged into polluted streams like the Monongahela, the Naugatuck, or the Blackstone.

Predatory Protozoa and other microscopic animals which feed on bacteria cause their reduction in natural streams.[7g]

As in ground waters, the number, not the mere presence, of *Bact. coli* is an index of the degree of pollution. Almost every river water will give some positive tests for *Bact. coli* with 100-c.c. test volumes. Table 203 by J. W. Ellms, shows fluctuations in bacteria and *Bact. coli* and the relations between the bacteria in Ohio River, a muddy stream containing on the average 135 p.p.m. of suspended matter. Table 204 gives similar results for the clearer but more highly colored Delaware River, containing 36 p.p.m. of suspended matter.[46]

Few of the colon type of bacteria grow better in warm water than in cold. Houston (1911)[7h] showed that while at 0°C. 46 per cent. of typhoid bacteria survive one week, only 0.07 per cent. survive at 10°C., and Ruediger[47] (1911) has shown that *Bact. coli* are far more abundant in the Red Lake River during the ice-bound winter period than in summer.

Lake waters contain fewer bacteria than river waters, although polluted with sewage at the shores and at the mouths of rivers. In 1897 Weston found in Lake Superior near Duluth no bacteria at a depth of 100 ft., although the surface water, polluted by sewage from Duluth and West Superior, Wis., contained several hundred per c.c. Dr. E. Channing Stowell found the water of Dublin pond (N. H.) practically sterile at a depth of 40 ft., although *Bact. coli* were occasionally found along the shores. Table 196 illustrates the variations in bacteria and *Bact. coli* in Crystal Lake, Wakefield, Mass., for the year ending Dec. 31, 1923. This is an example of a water which is slightly polluted. In this lake, the organisms growing at 37°C. numbered as follows: average bacteria per c.c., 251; maximum, 480; minimum, 126. The water is aerated and filtered before use.

Table 196. Bacteria in Crystal Lake Water, Wakefield, Mass.
Jan. 1–Dec. 31, 1923

Month	No. of bacteria on gelatine at 20°C.						Bact. coli		
				Variations in numbers No. of test days			10 c.c. tests		
	No. of test days	Mean per c.c.	Median per c.c.	100–300	300 1,000	1,000–10,000	Total No.	No. +	Per cent. +
Jan.	2	555	555	......	2	......	2	0	0
Feb.	1	280	280	1	......	......	1	0	0
Mar.	3	440	120	......	3	......	3	0	0
April	4	600	595	......	4	......	4	0	0
May	3	470	470	......	3	......	3	0	0
June	4	428	430	......	4	......	4	0	0
July	3	370	370	......	3	......	3	0	0
Aug.	4	511	465	......	4	......	4	0	0
Sept.	3	607	520	......	3	......	3	0	0
Oct.	4	715	530	......	3	1	4	0	0
Nov.	5	818	685	......	4	1	5	0	0
Dec.	4	754	715	......	4	......	4	0	0
Total	40	......	......	1	37	2	40	0	......
Average	......	546	478	......	......	......	......	......	......
Per cent. time	......	......	......	3	92	5	......	0	......

Lake Ontario near Toronto is an example of a polluted lake water. Results of examination of this water at Toronto for 1920 are given in Tables 197 and 198.

Table 197. Showing Fluctuation in Numbers of Bacteria and Bact. Coli per c.c. in Lake Ontario at Toronto Water Works, 1920

Norman J. Howard

Month 1920	Average no. of bacteria per c.c. on agar at 37°C	Average no. of Bact. coli per c.c.
January	610.7	8.32
February	474.2	7.42
March	337.0	11.01
April	368.8	19.13
May	554.7	36.07
June	779.3	54.34
July	1893.6	662.09
August	1644.6	2126.13
September	1525.2	690.28
October	885.9	23.22
November	912.6	96.12
December	598.7	191.33
Mean	830.4	327.11

Table 198. Yearly Percentage of Samples Showing Bact. Coli in Lake Ontario Water at Toronto Water Works, 1912-1924

Norman J. Howard

Year	Cubic centimeters								
	100	10	1	0.1	0.01	0.001	0.0001	0.00001	0.000001
1912	78.8	50.3	18.3	3.2	0.00	0.00	0.00	0.00	0
1913	81.2	54.6	24.6	4.9	0.00	0.00	0.00	0.00	0
1914	87.4	61.0	30.7	8.6	0.99	0.00	0.00	0.00	0
1915	91.4	60.2	31.9	8.2	1.60	0.30	0.00	0.00	0
1916	96.4	68.7	35.8	10.5	0.98	0.00	0.00	0.00	0
1917	92.7	67.1	32.2	8.5	2.60	0.00	0.00	0.00	0
1918	94.7	66.2	40.0	15.5	3.90	0.66	0.00	0.00	0
1919	98.0	83.1	53.0	28.4	12.50	4.00	0.33	0.33	0
1920	96.7	83.3	60.1	37.9	18.60	8.17	2.94	1.31	0
1921	98.7	93.1	72.7	41.8	22.00	9.90	3.00	1.00	0
1922	98.7	88.1	65.0	39.6	17.2	5.6	1.3	0.7	0
1923	95.4	83.2	55.3	29.9	13.5	3.3	0.33	0.33	0
1924	93.4	71.5	47.8	25.9	10.2	3.9	0.33	0.0	0

Suspended Mineral Matter. Most streams carry more or less suspended mineral matter, varying from several pounds to several tons per Mg. Quantities of suspended matter in various river waters are given in Table 207 (samples 1 to 69) (p. 621) and Table 23 (page 63). See also "The Industrial Utility of Public Water Supplies in U. S." by Collins, (W. S. Paper No. 496, 1923; and "Industrial Water Supplies," Foulk, Ohio Geol. Survey, 1925.

*Odors in surface waters** are due to: (1) *Organic Matter Other Than Living Organisms.* Peaty, straw-like, swamp-like, or marsh-like odor, usually

* See Ellms' investigation of tastes and odors at Cleveland, *E. N. R.*, June 16, 1921, p. 1039.

grouped under the term "vegetable," is caused by vegetable matter, mostly in solution. Wastes from paper and textile mills, bleacheries, dye houses, gas works, chemical works, tanneries, etc., often impart characteristic odors. (2) *Decomposition of Organic Matter.* Moldy, musty, unpleasant, disagreeable, or offensive odors caused by decomposition are quite readily recognized. Moldy odor, suggesting a cellar in which vegetables are stored, is frequently observed in water from old and badly polluted, but not necessarily infected, wells. Musty odor occurs in a sewage polluted water. Vegetable matter, unless decomposed, rarely produces anything worse than an unpleasant odor. The odor called disagreeable is usually from animal sources, while the odor termed offensive may be produced by sewage or by decomposition of Cyanophyceæ. (3) *Living Organisms.* Natural odors caused by organisms are produced by volatile, oily compounds secreted by the organisms, the most notable one being set free at the time the organism disintegrates. When dead organisms decay and suffer bacterial decomposition, odors different from the natural odors are produced. These are all offensive. Cyanophyceæ produce a "pig pen" odor upon decomposition (see below under Ice). Whipple[40e] stated that Synura "oil" may be detected in dilutions of 1.25,000,000.

Table 199. Number of Organisms per C.C. Required to Produce Noticeable Odor
(Whipple)

Organism	Description of odor	No. per c.c.
Synedra	Vegetable and earthy	5,000
Cyclotella	Aromatic and fishy	5,000
Melosira	Earthy and vegetable	3,000
Anabæna	Rank vegetable	17
Scendesmus	Vegetable and aromatic	25,000

The numbers of certain organisms per c.c. required to produce a noticeable odor are given in Table 199. Of 71 supplies taken from Massachusetts ponds and reservoirs, 45 were found to have given trouble, and 30 of these serious trouble, because of bad tastes or odors. Microscopic organisms are believed at present to be harmless. Many, however, are objectionable from an æsthetic standpoint and are often indicative of abundant food material emanating from objectionable sources. Distinctive odors produced by certain organisms may be described by the three general terms "aromatic," "grassy," and "fishy," as tabulated in Table 200.

Ice is always purer than the water from which it is gathered. Winslow has shown that typhoid fever bacilli die rapidly in ice, and that ice stored for 5 months is safe, even though it be collected from polluted sources. Table 201 gives analyses of water and ice from a pond in Massachusetts. The upper ice was polluted but not from the water. The water is not safe to drink but the ice may be used with impunity.

Growths of Oscillaria, Anabæna, and other organisms may become frozen in the ice of polluted ponds. Masses of these growths may decay and cause the formation of cavities within the ice containing dirty organic matter and gases which have a foul odor, often months after the ice is harvested.

Table 200. Distinctive Odors Produced by Certain Organisms[40c]

Whipple

Group	Organism	Natural odor
Aromatic odor..	Diatomaceæ:	
	Asterionella	Aromatic—geranium—fishy
	Cyclotella	Faintly aromatic
	Diatoma	Faintly aromatic
	Meridion	Aromatic
	Tabellaria	Aromatic
	Protozoa:	
	Cryptomonas	Candied violets
	Mallomonas	Aromatic—violets—fishy
Grassy odor....	Cyanophyceæ:	
	Anabæna	Grassy and moldy—green-corn—nasturtiums, etc.
	Rivularia	Grassy and moldy
	Clathrocystis	Sweet, grassy
	Cœlosphærium	Sweet, grassy
	Aphanizomenon	Grassy
Fishy odor.....	Chlorophyceæ:	
	Volvox	Fishy
	Eudorina	Faintly fishy
	Pandorina	Faintly fishy
	Dictyosphærium	Faintly fishy
	Protozoa:	
	Uroglena	Fishy and oily
	Synura	Ripe cucumbers—bitter and spicy taste.
	Dinobryon	Fishy, like rockweed
	Bursaria	Irish moss—salt marsh—fishy
	Peridinium	Fishy, like clam-shells
	Glenodinium	Fishy

Table 201. Analyses of Water and Ice from Same Source

Source of sample	Water	Ice	
		Cake	Upper 4 in.
Date of collection	Feb. 1922	Feb. 1922	Feb. 1922
Turbidity—silica standard	2	0	....
Color—platinum standard	50	0.0	....
Oxygen consumed	....	....	....
Nitrogen as free ammonia	0.018	0.040	....
Nitrogen as albuminoid ammonia (total)	0.210	0.066	....
Nitrogen as nitrites	0.002	0.004	....
Nitrogen as nitrates	0.240	0.030	....
Chlorine	11.4	0.00	....
Alkalinity	....	....	....
Hardness by soap method	35	....	....
Iron, Fe	0.70	....	....
Residue on evaporation (total)	98.0	20.0	....
Bacteria per c.c., 20°C	930	3	18
Bacteria per c.c., 37.5°C	80	4	22
Bact. coli in 1 c.c	Positive	Negative	Positive
Bact. coli in 50 c.c	Positive	Negative	Positive

Results except bacteria expressed in parts per million.

Examples of variation in suspended matter, as shown by turbidity, are given in Tables 202 and 204.

Table 202. Turbidity of Ohio River at Cincinnati (1924)*

1924 Month	No. test days	Mean turbidity	Median turbidity	Turbidity variations, (in heading) Number of test days, (in table) 0 to 10	11 to 25	26 to 50	51 to 100	101 to 250	251 to 500	Above 500	Suspended matter (gravimetric)
January......	31	370	320	0	0	0	0	9	14	8	472
February.....	29	240	135	0	0	0	3	19	2	5	280
March........	31	245	150	0	0	0	3	21	6	1	318
April.........	30	255	140	0	0	0	10	14	2	4	294
May.........	31	150	120	0	0	0	5	24	2	0	211
June.........	30	375	270	0	0	0	0	13	13	4	378
July..........	31	405	280	0	0	0	3	10	11	7	535
August.......	31	120	75	0	5	4	12	7	1	2	217
September....	29	200	110	0	1	1	10	10	5	2	284
October.......	31	155	45	0	9	7	4	2	7	2	294
November....	30	42	24	0	17	5	6	2	0	0	43
December....	31	335	180	0	2	4	1	10	5	9	273
Totals......	365	....	....	0	34	21	57	141	68	44	......
Averages...	......	240	150†	...	....	....	......	......	......	......	300
Per cent time.	99.8	....	....	0	9.3	5.7	15.6	38.6	18.6	12.0	......

* Compiled by Bahlman, Chief Bacteriologist, Cincinnati filtration plant.
† True median, not an average.

Table 203. Bacteria in Ohio River Water at Cincinnati for the Year 1924

Month	No. of bacteria on agar at 20°C., 48 hrs.									Bact. coli										B. coli* index, no. per c.c.
	No. of test days	Mean per c.c.	Median per c.c.	Variations in numbers; no. of test days						No. of test days	0.01 c.c. tests			0.1 c.c. tests			1.0 c.c. tests			
				0–100	101–300	301–1,000	1,001–10,000	10,001–100,000	above 100,000		Total No.	No. +	Per cent. +	Total no.	No. +	Per cent. +	Total no.	No. +	Per cent. +	
Jan.	31	27,600	26,000	2	1	1	9	17	1	31	31	6	19.4	31	16	51.6	31	31	100.0	21.0
Feb.	29	28,600	16,500	0	0	0	4	24	1	29	29	2	6.9	29	15	51.7	29	29	100.0	9.0
March	31	70,500	50,000	0	0	0	1	23	7	31	31	9	29.0	31	24	77.4	31	31	100.0	82.0
April	29	6,600	4,600	0	0	2	21	6	0	30	30	2	6.6	30	17	56.6	30	30	100.0	6.7
May	31	6,000	3,500	0	0	1	27	3	0	31	31	8	25.8	31	22	70.9	31	31	100.0	30.6
June	30	12,700	5,200	0	0	1	19	9	1	30	30	7	23.3	30	27	90.0	30	30	100.0	33.0
July	31	5,110	2,600	1	0	9	17	4	0	31	31	7	22.6	31	20	64.5	31	31	100.0	24.5
Aug.	31	3,760	370	2	11	11	2	5	0	31	31	3	9.7	31	9	29.0	31	23	74.2	12.0
Sept.	28	4,260	1,700	0	1	8	17	2	0	28	28	5	17.9	28	13	46.4	28	28	100.0	86.9
Oct.	31	11,600	3,000	1	4	6	11	9	0	31	31	4	12.9	31	17	54.8	31	28	90.3	14.8
Nov.	30	395	200	6	12	9	3	0	0	30	30	0	0.0	30	4	13.3	30	18	60.0	1.8
Dec.	31	57,700	38,000	0	2	3	2	21	3	31	31	9	29.0	31	23	74.2	31	29	93.5	33.7
Total	363	......	......	12	31	51	133	123	13	364	364	62	......	364	207	......	364	339	......	......
Average	....	19,500	5,200†	...	....	......	....	....	....	....	....	....	16.9	....	....	56.7	....	....	93.1	29.7
% time	99.2	......	......	3.3	8.5	14.0	36.6	34.0	3.6	99.45	99.45	17.0	......	99.45	56.9	......	99.45	93.1	......	......

* NOTE.—*Bact. coli* index is reciprocal of smallest test volume in which *Bact. coli* were found in a sample. It is an approximation of the number of *Bact. coli* present.

† True median, not an average.

Statistical forms recommended by Committee of N. E. W. W. A.

Table 204. Turbidity and Color of Delaware River Water for Year 1924

Parts per Million

Month	Num. of test days	Mean turbidity	Median turbidity	Turbidity variations, no. of test days: 0 to 10	11 to 25	26 to 50	51 to 100	101 to 250	251 to 500	Above 500	No. of test days	Mean color	Color variations, no. of test days: 0 to 10	11 to 20	21 to 50
Jan.	31	90	52	0	0	15	9	3	4	0	4	18	0	3	1
Feb.	29	65	33	0	2	18	3	6	0	0	4	14	0	4	0
Mar.	31	46	35	0	0	20	11	0	0	0	5	15	0	5	0
Apr.	30	52	28	0	13	11	3	2	1	0	4	14	1	2	1
May	31	36	28	0	9	19	1	2	0	0	4	16	0	4	0
June	30	30	26	0	15	12	3	0	0	0	5	18	0	4	1
July	31	31	27	0	9	20	2	0	0	0	4	18	0	3	1
Aug.	31	18	18	0	29	2	0	0	0	0	4	14	0	4	0
Sept.	30	15	15	1	29	0	0	0	0	0	5	13	0	5	0
Oct.	31	39	23	0	18	9	1	3	0	0	4	22	0	3	1
Nov.	30	26	25	0	16	14	0	0	0	0	4	16	0	4	0
Dec.	31	24	25	0	18	13	0	0	0	0	5	19	0	3	2
Total	366	14,376	...	1	158	153	33	16	5	0	52	197	1	44	7
Average	...	39	...	0.3	43.1	41.8	9.0	4.4	1.4	0	....	16	1.9	84.6	13.5
Results for Previous Years															
1924	366	39	...	0.3	43.1	41.8	9.0	4.4	1.4	0.0	52	16	1.9	84.6	13.5
1923	365	30	...	0.3	63.3	26.8	7.1	2.5	0.0	0.0	12	17	0	75.	25.
1922	365	35	...	0	54.0	35.1	7.1	3.3	0.5	0.0	52	15	17.3	76.9	5.8
1921	365	23	...	7.4	72.3	13.7	4.7	1.9	0.0	0.0	52	15	42.3	53.8	3.9
1920	366	24	...	44.8	35.2	9.8	6.6	2.2	1.4	0.0	52	26	0	.....	.....
1919	365	26	...	25.2	56.2	8.5	4.9	4.4	0.5	0.3	52	26	0	.....	.....
1918	365	24	...	38.1	44.6	8.5	5.5	2.2	1.1	0.0	53	18	0	.....	.....
1917	365	20	...	55.4	29.0	6.0	5.5	4.1	0.0	0.0	52	20	0	.....	.....
1916	366	16	...	50.8	38.3	7.1	2.7	1.1	0.0	0.0	53	19	0	.....	.....
1915	365	52	...	1.4	28.5	37.3	25.7	5.7	1.1	0.3	114	18	6	.....	.....
1914	365	20	...	43.5	40.0	11.0	1.9	3.3	0.3	0.0	52	15	0	.....	.....
1913	365	29	...	12.9	52.0	24.4	6.3	4.4	0.0	0.0	52	17	6	.....	.....

Table 205. Bacteria Recapitulation—Torresdale—Per C.C., on Agar at 37° C., after 24 Hr.
For Year 1924

Month	River		Applied to prefilters			Applied to final filters				* Effluent of filters				Filtered water basin†				Chlorine
								Per cent. removed				Per cent. removed				Per cent. removed		
	Num. of test days	Per c.c.	Num. of test days	Per c.c.	Per cent. removed by sedimentation basin	Num. of test days	Per c.c.	By pre-filters	From river	Num. of test days	* Per c.c.	By final filters	From river	Num. of test days	† Per c.c.	By chlorination	From river	Residual* P.P.M.
Jan.	31	596	31	240	59.7	31	86	64.2	85.6	31	8	90.7	98.7	31	5	37.5	99.16	.08
Feb.	29	549	29	261	52.5	29	147	43.7	73.2	29	17	88.4	96.9	29	6	64.7	98.9	.05
Mar.	31	545	31	298	45.3	31	138	53.7	74.7	31	13	58.1	97.6	31	7	46.2	98.7	.05
Apr.	29	623	30	456	26.8	30	109	76.1	82.0	30	6	94.5	99.04	30	5	16.7	99.20	.01
May	31	1,880	31	1,430	23.9	31	142	90.1	92.4	31	6	95.8	99.68	31	5	96.5	99.73	T
June	30	13,400	30	9,880	26.3	30	465	95.3	96.5	30	8	98.3	99.94	30	4	50.0	99.97	T
July	31	48,500	31	34,800	28.2	30	3,340	90.6	93.2	31	9	99.73	99.98	31	4	55.6	99.99	T
Aug.	31	54,300	31	35,800	34.1	31	3,540	90.1	93.5	31	13	99.63	99.98	31	3	76.9	99.99	.01
Sept.	30	31,000	30	23,200	25.2	30	2,570	88.9	91.7	30	10	99.61	99.97	30	2	80.0	99.99	.01
Oct.	31	5,960	31	5,010	16.0	31	685	86.3	88.5	31	6	99.12	99.90	31	3	50.0	99.95	.01
Nov.	30	1,160	30	1,040	10.4	30	178	82.9	84.7	30	10	94.4	99.14	30	2	80.0	99.83	.09
Dec.	31	778	31	703	9.7	31	168	76.1	78.4	31	21	87.5	97.3	31	6	71.4	99.23	.06
Total	365	4,890,040	366	347,323	....	365	351,385	.....	.....	366	3,825	.....	.....	366	1,561	.....	.....	9.43
Aver.	.....	13,400	.....	9,490	....	.....	962	.....	.....	.....	10	.....	.....	.....	4	.....	.....	.03
Per cent	.....	....	.....	....	29.2	.....	....	89.9	92.8	.....	.....	99.0	99.93	.....	.....	60.0	99.97	.....
Results for Previous Years																		
1924	365	13,400	366	9,490	29.2	365	962	90.3	92.8	366	10	99.0	99.93	366	4	60.0	99.97	.03
1923	364	14,900	364	14,500	2.7	362	1,410	91.0	90.5	353	21	98.5	99.86	362	4	80.9	99.97	.04
1922	.....	8,910	.....	6,200	30.4	.....	560	.....	93.7	.....	18	97.8	99.80	.....	5	72.2	99.94	.....
1921	.....	6,690	.....	....	....	.....	....	.....	.....	.....	11	.....	99.84	.....	3	72.7	99.96	.....

* Before chlorination.
† After chlorination.

Table 206. Bacteria of the Bact. Coli Group in Delaware River Water—Torresdale

For Year 1924

Month	.001 c.c. test			.01 c.c. test			0.1 c.c. test			1.0 c.c. test		
	No. of test days	Number +	Per cent. +	No. of test days	Number +	Per cent. +	No. of test days	Number +	Per cent. +	No. of test days	Number +	Per cent. +
Jan.	31	5	16	31	15	48	31	30	97	.....	.....	.....
Feb.	29	2	7	29	12	41	29	28	97	.....	.....	.....
Mar.	31	1	3	31	14	45	31	26	84	.....	.....	.....
Apr.	29	4	14	30	9	30	30	28	93	.....	.....	.....
May	31	1	3	31	19	61	31	31	100	.....	.....	.....
June	30	11	37	30	24	80	30	30	100	.....	.....	.....
July	31	12	39	31	28	90	31	31	100	.....	.....	.....
Aug.	31	18	58	31	28	90	31	30	97	.....	.....	.....
Sept.	30	18	60	30	27	90	30	30	100	.....	.....	.....
Oct.	31	12	39	31	19	61	31	31	100	.....	.....	.....
Nov.	30	5	17	30	22	73	30	29	97	.....	.....	.....
Dec.	31	6	19	31	22	71	31	31	100	.....	.....	.....
Total	365	95	.....	366	239	.....	366	355	.....	.....	.....	.....
Aver.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Per Cent of time	.....	.....	26.0	.....	.....	65.3	.....	.....	97.0	.....	.....	.....
Results for Previous Years												
1924	365	95	26.0	366	239	65.3	366	355	97.0	.....	.....	.....
1923	364	104	28.6	364	253	69.5	364	360	98.9	171	171	100
1922	364	98	27	364	280	77.0	364	361	99.2	364	364	100
1921	232	85	36.6	364	261	71.7	365	360	98.6	365	365	100
1920	.....	.....	.....	365	133	36.4	365	288	78.9	366	362	98.9
1919	.....	.....	.....	365	103	28.2	365	215	58.9	364	290	79.7
1918	.....	.....	.....	249	76	30.5	288	193	67.0	235	212	90.2
1917	.....	.....	.....	148	5	3.4	148	120	81.1	148	148	100
1916	.....	.....	.....	150	5	3.3	150	122	81.3	150	150	100
1915	.....	.....	.....	150	12	8.0	150	140	93.3	150	149	99.3
1914	.....	.....	.....	154	24	15.6	154	115	74.7	154	154	100
1913	.....	.....	.....	.....	.....	.....	153	131	85.6	153	152	99.3

Bact. coli communis reported previous to March, 1918; *Bact. coli* group reported thereafter.

Table 207. Water Analyses;* Chemical and Physical
(Parts in 1,000,000)

River Waters—American (Sanitary)

No.	River	Place	Reported by	Turbidity	Color	Oxygen consumed	Nitrogen as: Free ammonia	Nitrogen as: Albuminoid ammonia, Total	Nitrogen as: Albuminoid ammonia, Suspended	Nitrogen as: Nitrites	Nitrogen as: Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation: Total	Residue on evaporation: Suspended	Residue on evaporation: Dissolved	Iron	Carbonic acid
1	Allegheny	Pittsburgh, Pa.	Drake	85.0	6	3.20	0.060	0.150	—	0.006	0.22	17.0	48.0	19.0	220.0	—	—	2.0	—
2	Connecticut	Middletown, Conn.	Conn. State Bd. Health	—	30	—	0.068	0.164	....	0.008	0.18	2.7	32.0	....	74.0	....	....	....	—
3	Delaware	Philadelphia, Pa.	West	—	—	3.70	....	....	—	....	....	6.0	47.6	27.2	122.0	36	....	2.28	....
4	Hudson	Albany, N. Y.	Hazen	39.0	30	5.51	0.096	0.266	—	0.006	0.372	4.0	67.0	....	144.0	—	—	—	....
5	James	Richmond, Va.	Mason	....	....	1.65	0.550	0.150	—	tr.	tr.	1.17	—	....	105.0	—	—	....	....
6	Kennebec	Augusta, Me.	G. C. Whipple	5.3	32	....	0.019	0.125	—	0.001	0.04	1.06	19.1	15.3	57.8	....	....	—	—
7	Merrimac	Lawrence, Mass.	Mass. State Bd. Health	....	30	....	0.089	0.234	....	0.002	0.060	2.5	11.0	....	39.6	—	—	—	....
8	Mississippi	Minneapolis, Minn.	A. D. Meeds	—	40	7.60	0.072	0.240	—	tr.	tr.	2.0	164.0	150.0	197.0	—	—	—	....
9	Mississippi	New Orleans, La.	R. S. Weston	423.0	13	6.9	0.006	0.245	—	0.00	0.12	9.3	84.0	75.0	573.0	—	—	—	—
10	Missouri	Omaha, Neb.	Teichman	—	—	7.1	0.062	—	—	0.004	0.18	10.0	—	—	1325.0	—	—	—	....
	Ohio	Cincinnati, Ohio	G. W. Fuller:																
11	Maximum			—	—	46.0	0.074	0.868	0.758	0.030	1.34	—	57.0	70.0	2556.0	2333	223	—	47
12	Minimum			—	—	13.0	0.008	0.106	0.022	0.000	0.37	—	11.0	20.0	91.0	24	67	—	6
13	Normal			—	—	24.0	0.025	0.290	0.200	0.003	0.60	—	33.0	45.0	350.0	230	120	—	26
14	Potomac	Washington, D. C.	R. S. Weston	80.0	5	2.60	0.008	0.111	....	0.002	0.11	4.0	—	—	140.0	45	95	—	—
15	Schuylkill	Philadelphia, Pa.	Mason	—	—	—	0.010	0.100	....	0.000	0.46	—	—	—	133.4	—	—	—	—
15A	Missouri	St. Louis†	1925 report	4,700	36	41.5	0.048	1.480	—	0.006	1.36	4.0	92	—	—	—	—	—	—
15A	Missouri	St. Louis‡	1925 report	1,290	25	13.1	0.040	0.720	—	0.006	0.40	15	155	—	—	—	—	—	—

* See Standard Methods, Am. Pub. Health Assn., 1925, and *Water Supply & Irrigation Paper*, 560C., 1924 (Collins and Howard).
† High River stage at Market St., 26.3.
‡ Low River stage at Market St., 5.6.

River Waters—Foreign (Sanitary)

No.	River	Place	Reported by	Oxygen consumed	Nitrogen as: Free ammonia	Nitrogen as: Albuminoid ammonia, total	Nitrogen as: Nitrites	Nitrogen as: Nitrates	Chlorine	Hardness by soap	Residue on evaporation, total
50	Oder	Breslau	Poleck	15.4	0.06	—	—	1.2	7.0	—	135
51	Rhine	—	Vohe	—	—	—	—	tr.	2.5	—	250
52	Spree	Berlin	Tiemann	7.52 / 10.88	0.2	0.3	0.00	0.11	21.3	—	172 / 233
53	Thames	Sudbury, England	Frankland	2.01	0.006	0.330	—	2.10*	7.73	204	280

* This figure represents nitrites + nitrates.

Table 207. Water Analyses; Chemical and Physical.—(*Continued*)

River Waters—American (Mineral)

No.	River	Place	Turbidity*	Coefficient of fineness†	SiO_2	Fe	Ca	Mg	Na+K	NO_3	CO_3	HCO_3	Cl	SO_4	Residue on evaporation: Suspended	Residue on evaporation: Dissolved
16	Alabama	Selma, Ala.	141	0.72	21.0	0.53	13.0	2.9	7.0	0.7	0.0	48	2.3	9.0	100.0	82
17	Allegheny	Kittanning, Pa	21	1.14	7.9	0.13	14.0	3.0	11.0	0.7	0.0	38	14.0	17.0	30.0	87
18	Androscoggin	Brunswick, Me.	—	—	9.0	3.2	7.4	1.1	35.0	...	...	...	2.3	12.0	—	42
19	Arkansas	Little Rock, Ark.	755	0.88	28.0	0.82	55.0	13.0	144.0	2.0	0.0	148	203.0	93.0	748.0	630
20	Chattahoochee	West Point, Ga.	185	0.71	20.0	0.47	4.8	0.8	7.7	0.7	0.0	23	2.1	4.5	136.0	52
21	Colorado	Austin, Texas	...	...	18.0	3.10	52.0	17.0	49.0	...	0.0	195	59.0	42.0	351.0	321
22	Delaware	Lambertville, N. J.	16	1.74	9.0	0.07	12.0	3.3	5.4	1.1	0.0	46	2.9	12.0	26.0	70
23	Hudson	Hudson, N. Y.	13	1.26	11.0	0.15	21.0	3.8	7.9	0.8	0.0	73	4.0	16.0	16.0	108
24	James	Richmond, Va.	90	0.96	18.0	0.50	14.0	3.0	6.7	0.3	0.0	60	2.3	7.1	71.0	89
25	Mississippi	Minneapolis, Minn.	10	[illegible].13	15.0	0.07	40.0	14.0	10.0	1.4	0.0	188	1.6	18.0	7.9	200
26	Mississippi	Memphis, Tenn.	556	0.97	24.0	0.61	36.0	12.0	19.0	1.7	0.0	129	8.6	43.0	634.0	115
27	Mississippi	New Orleans, La.	—	—	11.0	0.53	32.0	8.4	13.0	2.5	0.0	111	9.7	24.0	—	71
28	Missouri	Kansas City, Kan.	1909	1.04	37.0	0.73	62.0	18.0	44.0	2.2	0.0	202	13.0	135.0	2032.0	426
29	Potomac	Cumberland, Md.	28	1.59	8.2	0.14	24.0	4.6	9.0	0.9	0.0	36	6.4	58.0	29.0	130
30	Rio Grande	Loredo, Texas	2475	...	29.0	3.6	104.0	23.0	119.0	...	0.0	178	164.0	228.0	...	791
31	Sacramento	Sacramento, Calif.	66	1.55	28.0	0.43	13.0	6.7	13.0	0.64	0.0	74	5.4	17.0	102.0	113
32	St. Lawrence	Ogdensburg, N. Y.	45	...	6.6	0.05	31.0	7.2	6.3	0.3	2.9	116	7.7	12.0	...	134
33	Tennessee	Knoxville, Tenn.	204	0.81	25.0	0.54	22.0	4.3	8.2	0.8	0.0	86	9.6	6.4	156.0	122

No. 16–33 Reported by U. S. Geological Survey, *Water Supply Papers*, 79, 236, and 237. * See also Table 22, p. 63.

River Waters—Foreign (Mineral)

No.	River	Place	SiO_2	Ca	Mg	Na	K	NO_3	CO_2	Cl	SO_4	Total residue
34	Danube	Vienna	5	34	7	...	...	...	61	5	13	141
35	Elbe	Hamburg	5	28	1	...	...	...	45	5	7	127
36	Garonne	Toulons	40	26	1	6	3	...	45	2	8	136
37	Loire	Orleans	41	19	2	9	6	...	42	3	2	134
38	Rhine	Basle	2	55	5	1	...	...	86	2	15	169
39	Rhone	Lyons	7	66	1	...	...	...	90	...	20	184
40	Spree	Berlin	...	26	3	2	2	3	45	...	9	114
41	Thames	London Bridge	2	82	...	14	2	...	69	64	32	408

No.	River	Place	CaO	MgO	SO_4	$Al_2O_3+Fe_2O_3$	Cl	KNO_3+NaNO_3	Total residue	Suspended residue	Volatile residue
42	Elbe	Surface	40	8.0	7	4	24	...	138	72	46
43	Elbe	Bottom	46	14.0	8	6	16	...	151	58	49
44	Oder	Breslau	29	8.1	14	...	7	1.2	135	...	...

No. 34–31, Stillman: Engr. Chemistry (Chemical Pub., Co. 1900). No. 42 and 43. Reported by J. Jettman No. 44. Reported by Poleck.

† See p. 594.

Table 207. Water Analyses; Chemical and Physical.—(*Continued*)
River Waters—Foreign (Mineral).—(*Continued*)

No.	River	Place	SiO_2	$CaCO_3$	$MgCO_3$	$CaSO_4$	$MgSO_4$	$K_2SO_4 + Na_2SO_4$	$Fe_2O_3 + Al_2O_3$	NaCl	Cl	$KNO_3 + NaNO_3$	$CaCl_2$	Total residue
45	Danube......	Vienna..............	4.9	83.7	15.0	2.9	15.7	2.0	2.0	1.0	5.8	....	....	126.2
46	Rhine........	Strassburg............	48.8	135.6	5.1	14.7	....	13.5	14.1	2.0	.....	3.8	....	231.8
47	Rhone.......	Geneva..............	23.8	78.9	4.9	46.6	6.3	7.4	3.7	1.7	.....	8.5	....	182.0
48	Seine........	Above Paris..........	......	92.0	39.0	20.0	*	*	8.0†	.....	.....	....	10.0	179.0
49	Spree........	Berlin...............	......	65.0	9.0	0.0	0.0	18.0	....	12.0	.....	3.0	....	114.0

* $MgSO_4 + K_2SO_4 + Na_2SO_4$ = 10.0. † Includes SiO_2. No. 45–49. From Philips Eng. Chemistry (Crosby, Lockwood and Son, 1902).

Table 207. Water Analyses; Chemical and Physical.—(*Continued*)
Rain and Snow; Springs and Wells; Ice

No.	Kind	Place	Reported by	Character	Oxygen consumed	Nitrogen as: Free ammonia	Nitrogen as: Albuminoid ammonia, total	Nitrogen as: Nitrites	Nitrogen as: Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation, total	CO_2	SiO_2
69	Rain.........	Lawrence, Mass...	Mass. State Bd. Health Report 1890.........	---	....	0.298	0.0024	0.0000	0.000	0.007	......	......	......	....	....
70	Rain.........	Jamaica Plain, Mass............	"	---	.....	0.056	0.0152	0.0004	0.018	0.130	......	......	......	....	....
71	Rain.........	London, Eng......	Mason..........	---	0.99	0.50	0.22	......	0.07*	6.30	......	......	39.5	....	....
72	Snow.........	Boston, Mass......	Mass. State Bd. Health Report 1890.........	---	.....	0.025	0.0038	......	0.003	......	......	......	......	....	....
73	Snow.........	Troy, N. Y.......	Mason..............	---	.....	0.046	0.022	0.00	0.00	0.188	......	......	......	....	....
74	Snow.........	London, Eng......	Mason..............	---	.....	0.06	0.021	0.024	0.02	1.08	......	......	16.0	....	....
75	Dirty cistern water.......	West Troy, N. Y..	Mason..............	Bad	2.25	1.050	0.175	Strong trace	0.00	2.00	......	......	20.0	....	....
76	Shallow well..	Elgin, Ill.........	Ill. State Water Survey.	Good	0.6	0.076	0.058	0.000	2.00	6.	......	210.0	298.0	....	....
77	Shallow well..	Elgin, Ill.........	Ill. State Water Survey.	Bad	2.0	0.032	0.152	0.002	8.00	200.	......	388.0	973.0	....	....
78	Deep well....	Champaign, Ill....	Ill. State Water Survey.	Good	4.3	1.600	0.160	0.000	0.08	3.	......	378.0	398.0	....	....
79	Deep well....	Moosup, Conn....	Conn. State Bd. Health.	Bad	0.24	0.028	0.038	0.019	11.2	17.7	142.0	44.0	229.0	....	....
80	Artesian well.	Pomfret, Conn....	R. S. Weston..........	Good	0.040	0.010	0.026	0.003	0.020	2.62	65.7	64.5	123.0	....	....
81	Artesian well.	Gloucester, Mass..	R. S. Weston..........	Bad	8.67	36.06	12.84	0.040	10.4	1200.0	335.0	1.9	2770.0	....	....
82	Dune........	Amsterdam, Hol..	Mason-Eng. News......	Good	......	0.19	0.106	0.0	0.33	32.3	......	......	344.0	96.8	7.6
83	Spring.......	Portland, Conn...	Conn. State Bd. Health.	Good	0.1	0.004	0.008	0.0	0.1	3.04	32.0	20.0	56.0	....	....
84	Spring.......	Brooklyn, Conn...	Conn. State Bd. Health	Bad	0.34	0.024	0.044	0.011	4.8	20.8	99.0	......	202.0	....	....
85	Ice pond.....	Wellesley, Mass...	R. S. Weston..........	Bad	......	0.024	0.218	0.008	0.2	9.5	35.1	29.0	96.0	....	....
86	Ice..........	Wellesley, Mass...	R. S. Weston..........	Good	......	0.078	0.104	0.007	0.00	0.5	9.5	1.0	12.0	....	....
87	Ice pond.....	Danvers, Mass....	R. S. Weston..........	Bad	5.82	0.026	0.200	0.001	0.70	18.60	40.8	24.6	101.0	....	....
88	Ice..........	Danvers, Mass....	R. S. Weston..........	Good	0.60	0.124	0.080	0.001	0.00	0.64	1.6	0.0	20.0	....	....

* This figure represents nitrates and nitrites.

Table 207. Water Analyses; Chemical and Physical.—(*Continued*)

Lake and Reservoir Water

a-American (Sanitary)

No.	Lake	Reported by	Turbidity	Color	Oxygen consumed	Nitrogen as: Free ammonia	Albuminoid ammonia, total	Nitrites	Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation, total	Iron	Sulfate	CaO	MgO
54	Champlain	U. S. Geol. Survey	—	—	—	0.031	0.132	0.002	0.092	0.9	50.6	—	65.6	....	—	—	—
55	Cochituate	Mass. State Bd. Health	....	24	4.2	0.029	0.221	0.001	0.027	5.4	20.0	—	52.1	....	—	—	—
56	Croton	Report of Comm. on additional water supply for N. Y.	6	27	—	0.077	0.191	0.002	0.10	1.7	39.4	31.0	71.0	0.23	—	—	—
57	Erie	Mason	....	..	1.25	0.045	0.112	tr.	0.08	3.5	....	....	134.0	....	—	—	—
58	George	N. Y. State Bd. Health	clear	..	2.0	0.066	0.062	0.001	0.04	1.0	12.7	6.0	38.0	....	—	—	—
59	Michigan*	Gehrman	—	00	7.1	0.024	0.044	0.00	0.200	5.0	93.0	....	140.0	0.4	—	—	—
60	Superior†	Mason	....	..	1.15	0.03	0.02	0.00	0.00	2.0	....	....	54.0	....	—	—	—
61	Wachusett Reservoir	Mass. State Bd. Health	....	18	3.1	0.022	0.161	0.001	0.024	0.25	8.0	....	30.3	....	—	—	—
62	Winnepesaukee	U. S. Geol. Survey	....	10	....	0.002	0.093	0.000	0.038	1.2	....	....	20.9	....	—	—	—

b-Foreign (Sanitary)

No.	Lake	Place	Reported by	Turbidity	Color	Oxygen consumed	Free ammonia	Albuminoid ammonia, total	Nitrites	Nitrates	Chlorine	Hardness by soap	Alkalinity	Residue on evaporation, total	Iron	Sulfate	CaO	MgO
63	Geneva	....	"Das Wasser"	....	..	....	....	....	....	....	....	....	....	128.0	....	38.1	42.3	11.5
64	Katrine	Scotland	Frankland	....	..	1.86	0.00	0.110	....	0.04§	6.7	9.5	....	29.6	....	....	....	....
65	Mügel	Berlin	Wehner	....	..	3.87	0.28‡	....	0.00	0.00	5.6	126.0	....	243.0	....	16.8	....	....
66	Tegel	Berlin	"Das Wasser"	....	..	12.9	0.70‡	....	....	2.1	12.6	....	....	188.3	....	10.2	63.5	81.0
67	Thirlmere	Manchester, Eng. Supply	Frank-and	....	..	1.74	0.080	0.170	0.00	0.00	7.5	9.0	....	23.6	....	....	....	....
68	Zürich	....	"Das Wasser"	....	..	....	....	....	....	....	....	....	....	140.6	....	9.2	56.6	10.0

* Four miles out at Chicago Intake.
† Near Duluth.
‡ These figures represent free ammonia + albuminoid ammonia.
§ This figure represents nitrites + nitrates.

Table 207. Water Analyses; Chemical and Physical.—(*Concluded*)

Sea Water

No.	Sea	Reported by	Nitrogen as: Ca	Mg	Na	K	NH_4	Al	Fe	Mn	CO_3	SO_4	Cl	Br	Residue on evaporation, total
89	Atlantic Ocean	Stillman Eng. Chemistry	557	1,198	17,720	668	....	....	....	....	....	3,030	20,840	388	38,400
90	Pacific Ocean	Stillman Eng. Chemistry	475	1,471	10,233	634	....	....	....	....	....	2,827	19,321	239	35,200
91	Baltic Sea	Stillman Eng. Chemistry	36	612	5,894	....	....	....	....	....	....	719	10,386	....	17,710
92	Black Sea	Stillman Eng. Chemistry	131	662	5,512	98	....	....	127.0	....	248	1,250	9,575	5	17,608
93	Caspian Sea	Stillman Eng. Chemistry	192	410	1,444	140	....	40	....	....	77	1,337	2,738	....	6,296
94	Dead Sea	Stillman Eng. Chemistry	9,000	19,883	47,918	6,385	18	153	12.0	26	....	471	154,442	2,177	240,483
95	Mediterranean Sea	Stillman Eng. Chemistry	444	1,310	11,706	264	....	....	2.8	....	68	2,943	20,527	434	37,706

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CHAPTER 26

PROTECTION OF WATER SUPPLY SOURCES

SURFACE WATER SUPPLIES

Catchment areas should be kept free from pollution, and, to this end, rules and regulations for their protection have been promulgated by the various State Boards and Departments of Health. But such rules, no matter how stringent, are often of little avail. Inhabitants, campers, and motorists often show but little respect for rules, and little is gained by prosecution. To prosecute offenders successfully, it is necessary that the delinquent be identified and the offense traced directly and surely to him, with specific proof as to time and place, and the isolation of his particular offense from those of others unknown. Better results can be obtained through education and cooperation. Ownership and patrol of the entire watershed theoretically maintain the purity of the water. But experience has shown that trespassing occurs, and even the patrol may become careless and pollution will result.* The automobile has increased the difficulties.

The protection of watersheds, while desirable in all cases, does not guarantee a safe water. Ultimate reliance for the safety of the water must be placed on proper purification and disinfection.*

River Supplies. Purification of river water by filtration or otherwise should be supplemented by careful inspection of the catchment area, especially above the waterworks intake, which should be protected from all pollution or contamination. Storage reservoirs are always advantageous, and should be carefully inspected to minimize their pollution. Filter beds and filter sand should be guarded carefully to prevent accidental pollution, and all filtered water basins, suction wells, and pipes, should be tight to prevent contamination by infected ground water.[1] Distributing reservoirs contain water about to be used[2] and should be carefully guarded or covered. Passing trains are a source of danger, difficult to control.

Lake and Reservoir Supplies. Long storage coupled with careful inspection can produce a fairly safe water even from polluted sources, such as the Lake Cochituate, Croton,† and Thames River catchment areas. While modern communities are demanding water which is safer and better in appearance than can be obtained without filtration, inspection can accomplish much in reducing dangers of infection. At the Pequannock sources the city of Newark has established luncheon camps to lure motor picnickers away from dangerous stopping places. These camps are supplied with drinking water, fireplace, rubbish burner, and privies.

* See "The Romance of Water Storage" by G. A. Johnson, *J. A. W. W. A.*, Vol. 8, 1921, p. 291.

† Croton water has, however, for a number of years been treated with liquid chlorine put into the aqueducts at a place relatively near New York City. Many engineers who have studied this water believe it should be filtered for both sanitary and physical improvement.

Lake sources should be frequently inspected and inspection supplemented by numerous and regular bacteriological analyses.

Water traffic on the Great Lakes is a grave source of danger, and lake craft should be required to disinfect discharges from toilet rooms. This can be accomplished by heating with steam or by addition of disinfectants to the sewage before discharge. (See Metcalf and Eddy, "American Sewerage Practice," 1915, vol. III, p. 737.)

Whether *bathing, boating, or fishing* should be allowed in a reservoir should be determined largely by the period of storage elapsing before use. In lakes or reservoirs remote from the distribution system, boating and fishing may be permissible, but should be restricted by permits. The issuance of any permits presents many difficulties of administration. Bathing offers serious danger because of the chance of pollution by typhoid urine and feces, especially the former. So offensive is the thought of drinking water previously used for bathing that the practice should be prevented wherever practicable. Fishing and boating should be discouraged.

Importance of Time. From a sanitary standpoint, the discharge of many million bacteria by one typhoid patient into a water about to be distributed is far more serious than the discharge of the sewage of a city, if the contaminated water be stored for a month or two before use. This consideration should enter into all rules and regulations for the protection of a source of supply.

Rules and Regulations. Most existing health laws are cumbersome and unwieldy, and throw burden of proof upon the owner of a water supply. Public opinion often impedes protective legislation. Some legislatures, however, have given State Departments of Health* authority to prepare rules and regulations for the sanitary protection of each catchment area. This is a better plan than general legislation, aimed to cover all sources of supply. Many State Departments of Health publish a Sanitary Code, with chapters referring to water supply protection.

Rules and regulations of the Board of Water Supply of the City of New York, duly approved by the State Department of Health, promulgated under Chap. 665 of the Laws of 1915 for protection against pollution of the waters of Ashokan reservoir and tributary water-courses:

Definitions. "The City" shall mean The City of New York or its duly authorized representative. "Watercourse" shall mean any natural or artificial spring, stream, or channel of any kind, in which water flows continually or intermittently over any part of the watershed to one of The City's reservoirs. "Reservoir" shall mean any natural or artificial lake, pond, or reservoir within the limits of a watershed used by The City for its water-supply.

Rules and Regulations. Sewage containing human excreta. (1) No privy or receptacle of any kind for the deposit, storage, removal or disposal of human excreta shall be constructed, located, or maintained in such manner as to pollute or threaten the pollution of any watercourse or reservoir.

(2) All transportable receptacles for human excreta shall be provided with tightly fitting covers, which shall be securely fastened during removal and transportation, so that no portion of the contents can escape. A sufficient number of receptacles shall be provided, so that whenever one is removed an empty receptacle

* State of Mass. Acts of 1895, Chap. 488 and amendments.

can at once be substituted. The receptacles shall be thoroughly cleansed and disinfected, as often as may be found necessary in order to maintain proper sanitary conditions.

(3) In the absence of some manner of disposal of excreta specifically approved by the Board of Water Supply after due submission of plans thereof to said Board, such excreta shall be disposed of by burial in shallow trenches not more than 2 ft. deep, in places well removed from all watercourses, where the flatness of the ground, the character of the subsoil and the depth of the ground-water level will afford ample security against contamination of any watercourse or undue pollution of the ground-water and the soil.

(4) In approved locations and where conditions of soil and topography are favorable, water-flushed toilets and other inside plumbing may be used, if connected by approved watertight pipes within an approved sewage-disposal system.

House Slops. (5) House slops, sink wastes, laundry water or sewage of any kind shall not be thrown on the ground or discharged in such manner as to become offensive or to result in or threaten the pollution of a watercourse or reservoir. Whenever large quantities of polluting solid matter or polluted liquid are to be disposed of, application shall be made to the Board of Water Supply for the approval of a satisfactory method of disposal.

Garbage, Refuse, Compost, Manure and Dead Animals. (6) No garbage or putrescible refuse of any kind shall be thrown or discharged directly into any watercourse or reservoir, nor shall the disposal be such as to result in or threaten pollution.

(7) Compost heaps, or masses of fermented or decayed foods, vegetables, roots, grain, sawdust, leaves or other vegetable substances shall not be maintained in such manner as to result in or threaten pollution.

(8) No dead animal of any kind, nor any part thereof, shall be thrown into any watercourse or reservoir or buried in such manner as to form a source of pollution.

Stables and Slaughter-houses. (9) No stable, pigsty, chicken house, barnyard, hog yard, slaughter-house, hitching or standing place of horses and cattle or any other place where dung or urine accumulates shall be constructed, located and maintained so as to form a source of pollution.

Factory Wastes. (10) No filth, decaying or putrescible matter, waste product or polluted liquid from any factory, creamery, mill, tannery, garage or establishment of any other kind shall be discharged, drained or washed directly into any watercourse or reservoir. Application shall be made to the Board of Water Supply for the approval of a satisfactory method for the disposal of such objectionable matters.

Washing, Bathing and Swimming. (11) No animals of any kind shall be washed in any watercourse or reservoir owned by The City. No clothes or other articles shall be washed in any watercourse or reservoir.

(12) No bathing, swimming nor washing shall be done in any watercourse or reservoir owned by The City.

Cemeteries. (13) No interment shall be made in any cemetery or other place of burial in such manner as to result in pollution of a watercourse or reservoir.

Disposal Systems. (14) Every existing system for treating excremental matter, wastes or discharges of any kind from any dwelling, hotel, stable, garage, factory or other building, wherein such wastes or discharges constitute a source of pollution, shall be modified in a manner satisfactory to the Board of Water Supply.

(15) All new systems for the treatment of excremental matter, wastes and discharges of any kind shall receive the approval of the Board of Water Supply before such system is installed.

NOTE.—These rules were prepared especially for the Catskill Mountain watersheds, where the slopes are very steep, the soil generally very shallow and impervious, and the habitable lands along the streams so narrow in many places that rules with set distances (like those of Metropolitan Water & Sewerage Board) would be inapplicable.

GROUND WATER SUPPLIES

Ground water supplies should be inspected to see that there is no chance for contamination from surface sources. Some ground waters of unimpeachable quality become polluted after being drawn to the surface, due to defective pump-wells or suctions. Ground waters which become infected are usually dangerous for a longer period than are surface waters and therefore should be inspected with equally great care. Inspection of sources, and chemical and bacteriological examinations of samples should go hand in hand.

Safety of Ground Waters. In a sandy soil, like that on Long Island, probably all ground water found under natural conditions at a depth of more than 10 ft. is safe for drinking, provided it does not receive immediate subsurface pollution. Bartow[3] found that safety of Illinois ground water increased with depth. Tubular wells will yield safe water, but open or dug wells are too liable to pollution from above to be used in crowded districts. In several European cities, ground water is preferred to filtered river water. It should not be drawn too near the surface on account of sewage nor too near the sea on account of salt (see also p. 245).

Natural Purification. *Oxygen Requirement of Soil.* Because the purifying action in the soil requires oxygen, the discharge of contaminating matter on the surface in the vicinity of a ground-water supply must be intermittent if the purity of the ground water be maintained. This is the condition which occurs in nature, a period of aeration following each period of rainfall. In cities and towns, pavements and buildings prevent aeration of the soil and consequently the purification of the water thereby. Sometimes purification is disturbed or arrested to such a degree that even wide zones of soil of the right physical character fail to protect ground-water sources against contamination.

Efficiency of Natural Purification. The distance which a water must pass through the soil before becoming safe is important, and depends upon local conditions, such as rate of flow, character of soil, and degree of pollution. Again the work imposed upon the soil may exceed its capacity; aerobic bacteria may be deprived of oxygen and give way to anaerobic bacteria, a condition not so favorable to bacteriological purification of ground water. A well-aerated soil is much more efficient than a soil which has been robbed of its oxygen.

Before sinking wells at Wuhlheide and Heiligensee, Ditthorn and Luerssen[4] made a careful study of the permeability of the soil to bacteria. The object of the test was to ascertain how near a well could be to a broken sewer pipe without danger of contamination. They used *Bact. prodigiosus* for their experiments. The first of this series of experiments was made at Tegel See where one of the waterworks wells, which was driven to a depth of 177 ft., and whose strainers began at a depth of 121 ft., was set apart for this purpose. At a distance of 69 ft. from this well, a pipe 62 ft. long was driven into the ground and through it the bacteria were introduced into the soil below ground-

water level through a strainer on the bottom of the pipe. At this point the ground-water level was but a few meters below the surface. A particular distance, 69 ft., was chosen because some of the proposed wells were to be that distance from a sewage line. The soil was made up of gravel and sand a little finer on the average than good mortar sand. Bacteria were introduced into the side pipe and water was pumped in afterward, until the level was 3 to 4.5 ft. above ground-water level, forcing the water containing the bacteria into this soil and directly into the ground-water stream. The test was conducted for several days continuously and 380,000 gal. of water were pumped from the well daily. During the first 45 days, samples were taken from the well daily. During the first 11 days of the test, 61,000,000 bacteria were put into the side pipe. Bacteria were detected in the water drawn from the well on 10 different days—the first time, 9 days after the first injection of bacteria into the soil; the last time, 19 days after the last injection. There were two intervening periods, one of 5 and one of 6 days, during which no bacteria were found. About one bacterium in 40,000, according to computations, reached the well water. It was concluded that the dangerous bacteria are much more sensitive and die out sooner than *Bact. prodigiosus*, and therefore the distance between the well and side pipe afforded sufficient protection.

In a second series of experiments made at Müggel See, bacteria were introduced above the ground-water level. The well was 140 ft. deep; the top of

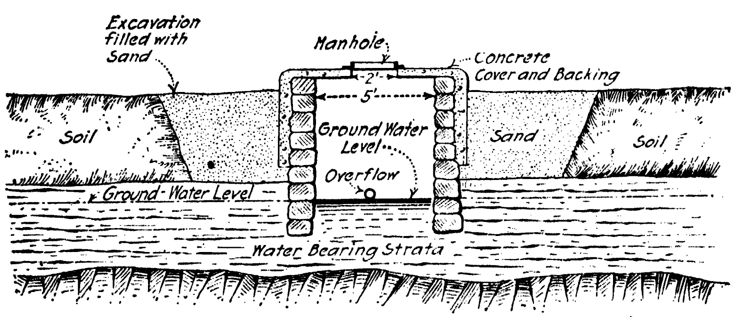
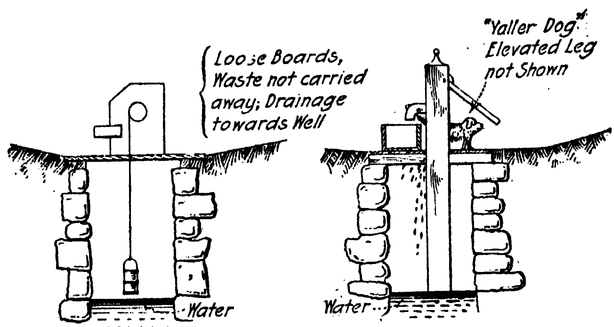


FIG. 249.—Restoration of a spring.

the strainer was 92 ft. below the surface and about 76 ft. below ground-water level, which in turn was about 16 ft. below the surface. The device for introducing the bacteria into the soil was an imitation broken drain. This was located 58 ft. from the well and was 7 ft. below the surface, therefore 9 ft. above ground-water level. The soil was a mixture of coarse and fine sand. The tests lasted 1½ months. Pumps drew an average of 126 gal. per hr. from the well. At the same time, 700 gal. of water per hr., on an average, were forced into the germ pipe. No positive results were obtained; then the germ pipe was lowered until it was only 4 ft. above ground-water level, and the quantity pumped from the well was increased to 3540 gal. per hr. In no case, however, were bacteria found in the well water. This test supports the general opinion that a wet zone is never so efficient a protection against pollution as a dry zone.

Sometimes a zone of soil 10 ft. wide will prevent the passage of contaminating material to a well, but ordinarily a greater width is essential; 50 ft. of fine sand nearly always suffices, provided ground-water level is 10 ft. or more below the surface. Where faults and fissures abound, as in limestone formations, contamination may pass a long distance through soil. At Lausanne, Switzerland,[5] the village well was polluted from a stream more than a mile away. Gaffky[6] reported the infection of an open well from a privy vault 50 ft. distant. Caution demands that a minimum of protection be avoided.*



Badly Constructed Wells

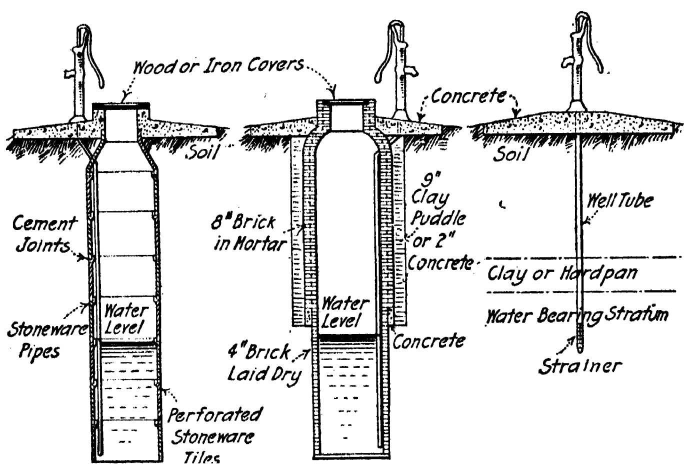


FIG. 250.—Satisfactorily constructed domestic wells.

Spring waters, which may be of high purity before emergence at the surface, may become polluted by surface drainage because of the conditions which surround the spring. In many cases such springs can be "restored" by following the rules given below for shallow wells, and by replacing the soil about the

* Stiles[6] found by experiments at an isolated army post that Bact. coli could be traced through sandy soil, in the direction of the ground water flow, 232 ft. from the point of pollution. The velocity of the ground water was less than one foot per day. In the same experiments, chemical pollution was traced 450 ft. by the use of uranin, an aniline dye, applied at the point of pollution. No Bact. Coli were found above the point of pollution.

spring for a depth of from 3 to 5 ft. or more, and for a width of 25 ft. or more, with a zone of fine sand. Figure 249 illustrates this method of protection.

Protection of Deep Wells. The lowering of the water-table by pumping directs the flow toward the well and invites polluting seepage. This pollution may enter the well if precautions are not taken to drive the bottom of the casing into the rock stratum to exclude the contaminating seepage. The tops of suction pipes should be sealed to prevent surface pollution from entering the well.

Protection of Shallow Wells.

In order to protect dug wells, the following rules should be observed:

1. Lining of well should be tight from surface of ground to water-bearing stratum; for a depth of 5 ft. in any case, better 10 ft.

2. Curb of well should be brought at least 1 ft. above surface.

3. Well should have a tight cover and the ground immediately around it should be raised so that surface drainage will be away from the well. Preferably the cover of the well should be of concrete or iron, and the ground about it paved with concrete or other impervious material (see also p. 234).

4. Pump discharge should be so arranged that waste water cannot return to well without passing through a layer of soil sufficient to protect the well against contamination.

5. There should be a zone of protection about each well in order to guard it against contamination from accidental polluting discharges in the vicinity of the well. This zone should have a width of at least 5 ft., preferably more.

6. Ventilation openings are usually not necessary. If used, they should be screened to keep out insects and small animals.

So-called "draw wells" with open tops are inferior to wells with pumps, and should be used only in exceptional cases. Wooden covers are likely to leak.

Conditions which surround tube wells are exactly the same as those which surround other wells of the same depth. Their advantage lies in the greater protection afforded by the tight well tube. The tubes themselves, however, dissolve rapidly in soft waters containing large amounts of free CO_2, thereby giving rise to leaks and making renewals frequent and expensive.

DISINFECTION

Disinfection of excreta and contaminated surroundings is always desirable. For this purpose, lime, calcium hypochlorite, metallic salts, and formaldehyde are used.

Numerous coal-tar products, many of them very efficient, are also used as disinfectants. Their strengths are given in terms of the "*Phenol Equivalent,*"* obtained by dividing the figure indicating the degree of dilution of the disinfectant that kills specific bacteria in a given time by that expressing that degree of dilution of pure phenol which kills the same organisms in the same time and under parallel conditions. *Soap* increases the efficiency of coal-tar disinfectants, and cleanliness is safer than inadequate disinfection.

Table 208. Disinfectants

Disinfectant	Solution for ordinary use, %	Proportion of chemical to disinfect excreta
Quicklime (CaO)	15.0	Equal volumes of excreta and 15 per cent. milk of lime.
Chlorinated lime (bleaching powder) (calcium hypochlorite).	4.0	Equal volumes of excreta and 4 per cent. solution.
Phenol (carbolic acid)*	5.0	Equal volumes of excreta and 5 per cent. solution, or 2.5 per cent. phenol in the final mass.
Cresol	2.0	One per cent. cresol in the final mass.
Formaldehyde, ("Formalin" = ± 30 per cent. solution of Formaldehyde).	5.0	2.5 per cent. formaldehyde in final mass.
Mercuric chloride ($HgCl_2$)	0.25 0.1	Equal volumes of excreta and 1:500 solution.
Zinc chloride ($ZnCl_2$)	10.0	Equal volumes of excreta and 10 per cent. solution.

* Crude carbolic acid has 2.75 times the germicidal potency of pure phenol.

Bibliography, Chapter 26. Protection of Sources

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* Some prefer the term "Phenol Coefficient."

CHAPTER 27

STORAGE OF WATER AND IMPROVEMENT OF RESERVOIRS

Quality of water is greatly affected by storage, both for good and bad. Storage affords opportunity for coagulation, precipitation, bleaching of color, death of exotic disease germs, and subsidence of silt and clay. Sometimes, however, stored waters develop growths of microscopic organisms, especially if the sources contain an abundant supply of food material or the water be stored in a new reservoir from which the vegetation and other organic matter have not been adequately removed. If the run-off be from a clean drainage area and the water be stored sufficiently long in an organically clean reservoir of adequate depth, a satisfactory supply usually results.

By *sedimentation or subsidence* waters are clarified although some waters containing colloidal clay are not clarified completely by subsidence for periods of practicable length. Some reduction in numbers of bacteria is caused by adsorptive effect of precipitating suspended matter. The effect of subsidence in the reservoir of the Washington, D. C., Works which are supplied with turbid water from the Great Falls of the Potomac, is shown in Table 209.

Table 209. Average Efficiency of Washington Water Purification System

Reservoir outlet	1907	1908	1909	1910	1911	1912	Average for 6 yrs.
Turbidity							
Dalecarlia	46	53	50	30	18	59	43
Georgetown	37	45	32	29	16	23	30
McMillan Park	29	31	22	18	10	13	22
Filtered water	2	2	1	1	0	0	1
Bacteria							
Dalecarlia	1,940	2,700	1,950	13,850	3,370	6,000	4,970
Georgetown	1,680	2,940	950	10,850	2,080	2,600	3,350
McMillan Park	635	1,250	390	6,820	1,390	1,100	1,930
Filtered water	31	55	21	143	38	35	54

Period of storage in natural lakes may be many years, as in the Great Lakes, and may result in complete clarification and decolorization. Silver Lake, Brockton, Mass., has a storage capacity of 1766 days or 1,579,800 gal. per sq. mi. of drainage area; the color of the water is reduced from about 100 to 9 p.p.m. Periods of storage in certain other reservoirs are given in Fig. 251. At least 90 days' storage should be provided where surface waters are used without other means of purification. Even this nominal period may not afford sufficient protection against pollution because of imperfect displacement of the reservoir contents due to differences in the density of the water, due in turn to temperature changes and to the action of wind and currents. It is too short to avoid growths of algae.

Storage ratio is the capacity of the reservoir divided by the mean yearly inflow.

Limnology, or the science of lakes and ponds, treats of their geology, geography, physics, chemistry, biology, and the mutual relations of these features. (See Whipple, "Microscopy of Drinking Water," 1914, Chap. VII.)

Lake Thermometry. For surface temperatures, an ordinary chemical thermometer may be used, graduated from 0° to 40°C. or from 20° to 120°F.

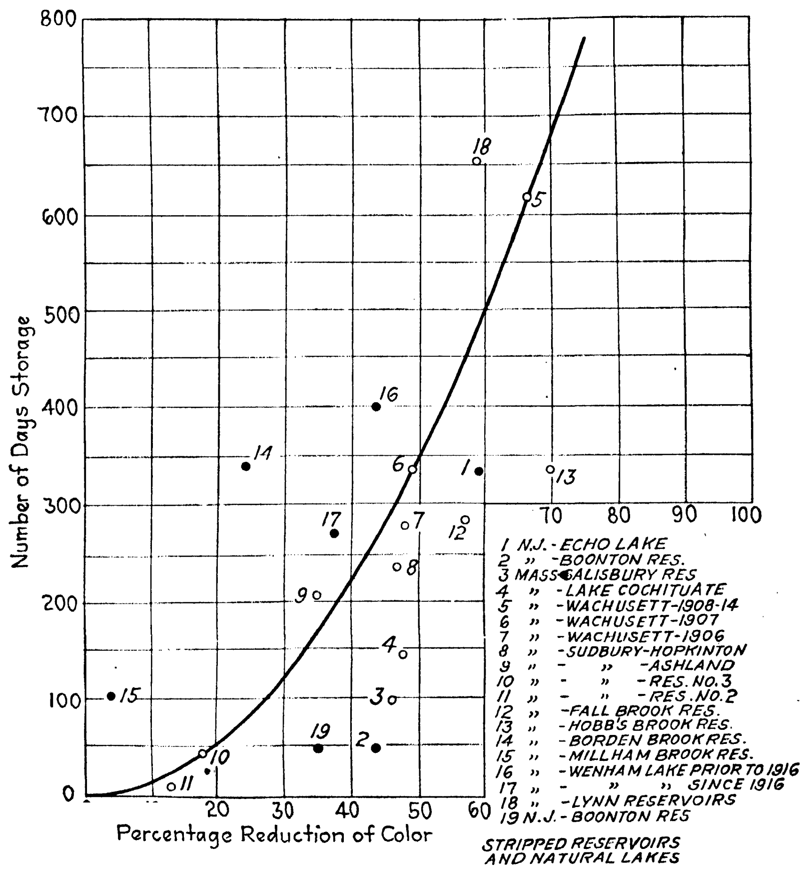


FIG. 251.—Reduction of color by reservoir storage.
(Weston and Sampson, 1924.)

For subsurface temperatures at depths of less than 50 ft., this thermometer may be put in a weighted case enclosed in a stoppered bottle, lowered to the proper depth and filled by withdrawing the stopper. After allowing sufficient time for the thermometer to set, the bottle is drawn to the surface and the thermometer read before it is taken from the bottle. For deeper, as well as for shallower, temperatures the *thermophone** of Warren and Whipple,[1a] an elec-

* Not stock apparatus. Must be made to order by makers of resistance thermometers.

trical thermometer of the resistance type, is better adapted than any other instrument for taking series of observations.

Circulation Due to Temperature. Because water is at its greatest density at 4°C. (39.2°F.) it follows that fluctuations in surface temperatures of lakes and ponds cause vertical circulation. As the surface water becomes cooler in winter, it becomes denser and sinks to the bottom. This overturn continues until a period of fairly stable equilibrium, known as winter stagnation, is reached. In the spring, the surface water begins to grow warmer and, until it reaches 4°C. grows denser and sinks. Therefore there is a period of circulation until all the water has reached the temperature of maximum density. These changes are known as the fall and spring circulations or overturns, respectively.

Lakes are divided into three types according to their surface temperatures and into three orders according to their bottom temperatures. The three types are *polar, temperate* and *tropical.* In lakes of the polar type, surface temperature is never above that of maximum density; in lakes of the temperate type it is sometimes below and sometimes above it; in lakes of the tropical type it is never below that point. The three orders may be defined as follows: Lakes of the first order have bottom temperatures which are practically constant at, or very near, maximum density; lakes of the second order have bottom temperatures which undergo annual fluctuations, but which are never very far from maximum density; lakes of the third order have bottom temperatures which are seldom very far from the surface temperatures.

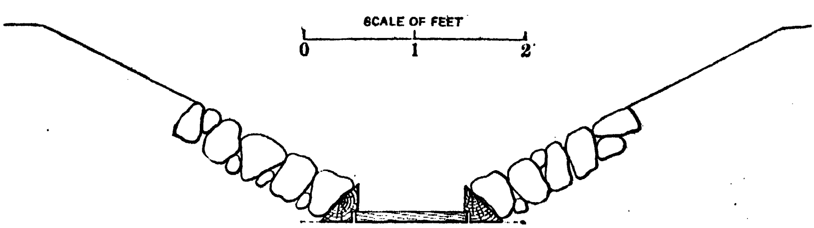
Table 210. Circulation Periods of Lakes[1b]

Kind of lake	Polar type	Temperate type	Tropical type
First order......	One circulation period possible in summer, but generally none.	Two circulation periods possible, in spring and fall, but generally none.	One circulation period, possible in winter, but generally none.
Second order.....	One circulation period, in summer.	Two circulation periods, in spring and autumn.	One circulation period, in winter.
Third order......	Circulation at all seasons, except when surface is frozen.	Circulation at all seasons, except when surface is frozen.	Circulation at all seasons.

Wind Currents. Wind produces important horizontal currents. The velocity of surface water in Lake Erie is about 5 per cent. of the air velocity. Ackermann showed that the surface water of Owasco Lake, N. Y., had a velocity of 3 per cent. of a wind velocity of 5 mi. an hr. and 1 per cent. of a wind velocity of 30 mi. an hr. After wind-driven water strikes the shore, it runs back below the surface. This is called an *undertow current.* Between the surface and the undertow currents there is a slow-motion zone called the shearing plane.

Removal of Color. Storage removes color because of the bleaching action of sunlight, and the coagulation and precipitation of colored vegetable matter

held in colloidal solution. The sun's rays are rapidly absorbed by water and their bleaching action is greatest at the surface. In clear waters there is little bleaching action below a depth of 5 ft., and in turbid waters the effect of sunlight may be felt only a few inches. Wachusett* records from 1907 to 1914 show that rate of reduction of color at bottom of reservoir was 30 per cent. less than at surface. In Ashland and Hopkinton reservoirs, Mass., bottom decolorization is about 50 per cent. of that at surface. In some waters coloring matter is in colloidal solution, for example, Dismal Swamp. Such coloring matter is often reduced quite suddenly, by the coagulation of fine particles and their rapid precipitation. *Sunlight* is not indispensable for the coagulation of colloidal color. *Iron* is a component part of chlorophyll and its derivatives, from which the coloring matter in water is derived. As the coloring matter is decomposed in the presence of oxygen, the iron is changed to its insoluble ferric state and is precipitated on the bottom of the reservoir. During the warmer months, the bottom water in deep reservoirs becomes robbed of its oxygen, the iron is reduced to the soluble ferrous state and enters into



SECTION OF SWAMP DRAINAGE DITCH

FIG. 252 [2]

combination with the organic matter. When this compound reaches the surface it oxidizes and increases the color of the water. Exposure to light, oxygen, and bacteria bring about a decomposition of the coloring matter and an oxidation and precipitation of the iron.

The degree of reduction in color for various periods of storage in reservoirs in the Northeastern United States is shown in Fig. 251.

Hydrogen sulfide is frequently produced by putrefaction of organic matter in the stagnant bottom water in a reservoir. This forms ferrous sulfide with the iron and imparts a black color to the water. Water from the bottoms of reservoirs containing excessive CO_2 or organic acids is apt to corrode lead pipes rapidly, making necessary the addition of lime and sometimes aeration.

Swamp drainage is an effective method of color prevention. Study of a watershed frequently determines certain swampy areas which are the principal sources of color. Desmond FitzGerald estimated that the color of the water supplied by the Ashland reservoir (Metropolitan Waterworks, Boston) could be reduced from 56 to 45.

F. P. Stearns used a simple form of *swamp drainage ditch* shown in Fig. 252. In connection with the Metropolitan Waterworks (Boston), 1069 acres of swamp were drained by 144,558 ft. of ditches.

* Metropolitan Waterworks, Boston.

Soil Stripping. Frequently, especially in Massachusetts, bottoms and sides of reservoirs are stripped of all growing vegetation and surface soil, and all deposits of muck are covered with a layer of clean sand. Waters stored in stripped reservoirs are greatly improved in color and in odor (due to organisms) especially during the first few years. Ralph H. Stearns[3] has studied the decolorization of water in certain stripped and unstripped reservoirs in Massachusetts. See *E. N.*, May 6, and Aug. 12, 1915, articles by Hazen and Whipple and F. P. Stearns. More data exist for stripped than for unstripped reservoirs. Ultimately the character of the bottom and sides of a reservoir will be controlled almost entirely by the accumulation of ooze precipitated from the water (see page 643).

Organic matter which has the greatest effect upon the quality of impounded waters is derived from grass, weeds, trees, and other vegetation on the reservoir site. They should be removed by cutting and burning just before the reservoir is filled. To reduce growths of aquatic weeds and filamentous algæ, the shores of the reservoir from 2 to 5 ft. vertically above the high-water mark and 10 to 20 ft. or more below, according to range of surface fluctuation, should be grubbed of stumps and roots. Below this zone, stumps should be pulled or cut close to ground. It is well to remove all deciduous trees which are so near the reservoir that leaves would fall therein, or to provide a screen of evergreens. Stripping undoubtedly reduces growths of organisms, but it is the result of experience that it does not entirely or uniformly eliminate unpleasant or offensive odors. This is shown by Massachusetts experience. Stripping alone cannot be relied upon to produce a stored water which will be satisfactory at all times, in taste, odor, or color. Modern standards call for a color less than 10 p.p.m. and no unpleasant odor at any time. Such a result can be obtained by other methods of purification, such as filtration. Where filtration is necessary, the expense of complete soil stripping is seldom warranted. After the first few years, no considerable portion of the cost of stripping can be saved through a resultant reduction in the cost of filter operation.

Origin and Longevity of Polluting Bacteria. To safeguard the purity of water supplies, careful and proper inspection should be made in order to eliminate any possible source of contamination. Chief sources of contamination are sewage of cities and towns, and wilful or careless discharge of dejecta by individuals. Water-borne diseases comprise those caused by bacteria carried from the intestine of one individual to the mouth of another. Intestinal bacteria, including *Bact. coli* and the typhoid bacilli, tend to die out in natural waters. According to various authorities (Gärtner,[4] Jordan, Russell and Zeit,[5] and Houston[6]) over 95 per cent. of typhoid bacilli discharged into surface water die during the week following their entrance, and after 1 month they have practically ceased to exist. Table 211 emphasizes this point. The surviving minority may persist for 2 months or longer. Experiments have shown that typhoid bacilli perish more rapidly in polluted surface waters than in unpolluted ground waters. Jordan, Russell, and Zeit[5] attribute this to the antagonism of normal water bacteria. Protozoa also consume bacteria.

Table 211. Vitality of the Typhoid Bacillus[6]

(HOUSTON)

Experiment	Initial number of typhoid bacilli per c.c. of the infected *raw* river water	Number of typhoid bacilli per c.c. of the infected *raw* river water, after storage in the laboratory for following periods, weeks					Number of weeks required to effect the destruction of the typhoid bacillus in 100 c.c. of the infected *raw* river water
		One	Two	Three	Four	Five	
1 T.....	40	0	......	....	......	......	Five
2 L......	40	0	......	....	......	......	Five
5 L......	170,000	53	2	0	......	......	Five
15 N. R..	525,000	29	3	0	......	......	Five
3 N. R..	40	0	......	....	......	......	Six
4 T.....	170,000	9	2	0	......	......	Six
							Six
6 N. R..	170,000	40	2	0	......	......	......
8 L......	470,000	850	11	7	2	0	Seven
9 N. R..	470,000	1,430	14	7	0	......	Seven
14 L......	525,000	32	2	0	......	......	Seven
18 N. R..	475,000	30	3	0	......	......	Seven
7 T.....	470,000	480	31	5	0	......	Eight
10 T.....	8,000,000	3,000	30	4	0	......	Eight
11 L......	8,000,000	2,900	29	5	0	......	Eight
13 T.....	525,000	12	1	0	......	......	Eight
17 L......	475,000	80	11	2	0	......	Eight
12 N. R..	8,000,000	400	22	2	0	......	Nine
16 T.....	475,000	210	12	2	1	0	Nine

T. = Thames, L. = Lee, N. R. = New River.

Longevity of Bact. Coli and B. Typhosus in Water. "In pure natural water and in redistilled water *Bact coli* and *B. typhosus* die from starvation at a regular rate. The rate of death increases with the temperature and is similar to the rate of a chemical reaction, thus following the monomolecular law. The presence of mineral matters had no apparent effect on the organisms. The presence of oxygen under starvation conditions seems to be harmful to *Bact. Coli* and beneficial to *B. typhosus.*" (Conclusions by M. E. Hinds).[7]

Effect of Storage on Microscopic Organisms. In order to prevent growths of organisms, water from the drainage area should be delivered quickly and stored in a reservoir which will not add to the organic content. It should not be allowed to stand for any considerable length of time in contact with organic matter. In deep reservoirs, little life exists below the transition zone.* Crenothrix and fungi, however, may develop in stagnant zones. Most growths of organisms occur near the surface where light is abundant. Vertical circulation causes distribution of algæ as well as of food for their growth. Under the influence of sunlight in the upper layers, algæ grow readily, and are held at the surface by the gases evolved during growth. *Growths of microscopic organisms* vary with the seasons as do flowering plants. In Northeastern United States diatoms exhibit a spring and autumn growth. Spring growth is usually at its height in May, autumn in September. Chlorophyceæ (green algæ) reach their maximum in summer (June and July), and are

* Transition Zone, sometimes called Thermocline or Discontinuity Layer—German: Sprungschicht— is the relatively thin layer between the upper and lower layers, where the temperature changes rapidly with the depth.

followed by Cyanophyceæ, which reach their maximum in October. The most troublesome forms of Cyanophyceæ, *e.g.*, Anabæna, rarely develop at a temperature below 60 to 70°F., although growths may linger until frozen into the ice. Seasonal distribution of Protozoa is very variable. Rotifera are most numerous in summer and growths of Crustaceæ, as a rule, reach their maximum in the spring.

Storage of ground waters should invariably be in the dark to prevent growths of microscopic organisms, especially diatoms. Of these, Asterionella is the most troublesome. Storage may not prevent the growth of fungi and certain Protozoa which do not require light for their existence. In a properly designed well system there is usually sufficient storage in the water-bearing strata.

Storage of Filtered Waters. Filtered waters, especially those high in mineral matter and carbon dioxide, have to be stored like ground waters. Certain soft waters from upland sources may be stored in open reservoirs with impunity.

Advantages of Covered Reservoirs. See page 538. (Bunker.) 1. To maintain the water in the same condition of purity in which it leaves the ground or purification plant by: (*a*) Preventing entrance of dust, bird droppings, leaves, etc. (*b*) Preventing growth of algæ. 2. To eliminate frequent cleaning. 3. To lessen danger of pollution by tresspassers. 4. To prevent loss of water by evaporation. 5. To maintain water at an even temperature. 6. Because first cost is cheaper than later remodeling, and causes no interruption of service.

Bibliography, Chapter 27. Storage

1. Whipple: "Microscopy of Drinking Water," Wiley, 1914. **a,** p. 87; **b,** p. 102. **2.** *T. A. S. C. E.*, Vol. 44, 1900, p. 176. **3.** *J. N. E. W. W. A.*, Vol. 30, 1916, p. 20. **4.** *Klin. Jahrb.*, Vol. 9, 1902, p. 335. **5.** *J. Inf. Diseases*, Vol. 1, 1904, p. 641. **6.** "Studies in Water Supply," 1913, p. 83 *et seq.* **7.** *Univ. of Illinois Bull.*, Oct. 2, 1916 (Vol. 14, No. 5).

CHAPTER 28

SUBSIDENCE§

Plain subsidence serves to remove the subsidable or settleable silt and clay from water. It is the cheapest method of removing those particles which settle out in a moderately short time and which would rapidly clog a filter. Subsidence is effected in open basins with concrete floors or in impounding reservoirs, designed to hold from a few hours' to several days' supply. In the *intermittent* system, water is passed into a basin, permitted to stand, and then drawn off. In the *continuous* system, water flows through the basin continuously. Cleaning is usually accomplished hydraulically by operating quick-opening gates and flushing out the sediment. Bottoms of well-designed subsiding basins usually slope to drains or openings. Hose streams are frequently used to move the sediment to the drains. At Grand Rapids, Mich.,[2] pressure-flushing for about ½ hr. through ⅛-in. holes in 2½-in. pipes laid on the floor of the basin, precedes the draining. A very few plants employ hydraulic dredges. Self-cleaning basins are not usually practicable because of the large size of some sediment particles, although sand ejectors have been built in the bottoms of grit chambers, as at Cleveland, and since the floods in the Mississippi River basin, in 1923, there has been a tendency to use mechanically cleaned basins of the Dorr type.

Theoretical Considerations. The theory of subsidence has been elucidated by Hazen[1a] and others. Every particle in suspension tends to move downward at a velocity depending upon its size, weight, and shape, and upon the viscosity of the water. Spherical particles settle more rapidly than irregular particles of the same specific gravity. Particles in colloidal solution are so fine that they are not acted upon by gravity unless coalesced into groups.

Table 212. Velocities at which Particles of Sand and Silt will Subside in Still Water

Diam. of particles, in mm.	Hydraulic value, in mm. per sec. at 10° C. = 50° F.	Diam. of particles, in mm.	Hydraulic value, in mm. per sec. at 10° C. = 50° F.
1.00	100*	0.03	1.3†
0.80	83*	0.02	0.62‡
0.60	63*	0.015	0.35‡
0.50	53*	0.010	0.154‡
0.40	42*	0.008	0.098‡
0.30	32*	0.006	0.055‡
0.20	21*	0.005	0.0385‡
0.15	15*	0.004	0.0247‡
0.10	8*	0.003	0.0138‡
0.08	6†	0.002	0.0062‡
0.06	3.8†	0.0015	0.0035‡
0.05	2.9†	0.001	0.00154‡
0.04	2.1†	0.0001	0.0000154‡

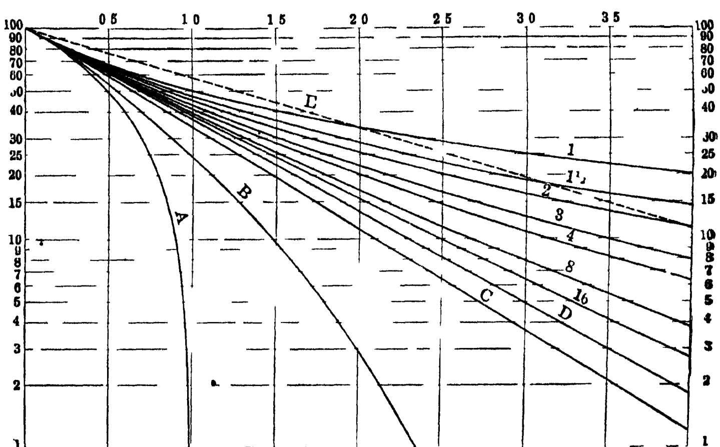
* Experiments by Hazen. † Interpolated from connecting curves. ‡ Wiley's formula.

§ For Atlanta basins, see *E. N. R.*, May 13, 1920, p. 958.

Table 213. Grades of Suspended Particles in Water

Kind of material	Diam. of particles, in mm	Kind of material	Diam. of particles, in mm
Coarse gravel	2 upward	Silt	0.05–0.01
Fine gravel	2–1	Fine silt	0.01–0.005
Coarse sand	1–0.5	Clay	0.01–0.001
Medium sand	0.5–0.25	Fine clay	0.0010–0.0001
Fine sand	0.25–0.1	Colloidal clay	Finer than 0.0001
Very fine sand	0.1–0.05		

Water containing sediment and free from colloidal matter (*e.g.*, clay), in a perfectly quiet basin will clear most rapidly at the top, the coarse particles will go down faster and there will be a progressive downwardly clearing. Most minerals suspended in water have a specific gravity of from 2.1 to 2.9. The specific gravity of quartzite sand is 2.65.



$\frac{a}{t}$ or Time of Settling in Terms of Time Required for One Particle to Settle from Top to Bottom

FIG. 253

Basin Types. When

t = time required for a particle of sediment to fall from surface to bottom of water in basin, water meanwhile being absolutely still,

a = time of sedimentation in case action is intermittent (in case of continuous operation, let a be quotient obtained by dividing capacity of basin by quantity of water entering or leaving it during each unit of time),

n = number of basins, in case several basins are used successively,

x = proportion of sediment remaining at end of process, amount at beginning being taken as unity,

$\frac{a}{t}$ = time of settling, in terms of "t,"

the values of x for various values of $\frac{a}{t}$ are plotted in Fig. 253 as line A.

This is the theoretical maximum and cannot be reached in practice because suspended matter does not subside in the manner indicated. Its

subsidence is interfered with by the vortex motion produced by the kinetic energy of the entering water, by the action of the wind, by convection currents produced by temperature, and by mixing produced by currents caused by the shape of the basin.

Table 214. Subsiding and Coagulating Basins[1b]

Basin	Area, sq. ft.	Average, depth, ft.	Approximate horizontal course, ft.	Ratio of horizontal course to depth	Quantity treated daily, gals.	Hours of storage	Gals. per sq. ft. daily
East Jersey	5,400	43.0	130	3	32,000,000	1.30	5,920
Jewell, Pittsburgh	131	6.7	......	.....	250,000	0.63	1,910
Warren, Pittsburgh	176	10.0	......	.....	300,000	1.05	1,700
†Detroit, Mich	252,000	15.9	1000	63	350,000,000	2.0	1,387
Ithaca	4,700	11.4	240	23	3,000,000	3.20	638
Watertown	9,880	13.6	270	20	6,000,000	4.00	610
†Cleveland, O.	192,042	14.6	500	34	150,000,000	3.36	782
†Baltimore, Md	147,700	18.0	700	39	128,000,000	3.0	868
World's Fair, Chicago, 1893 sewage	3,136	40.0	......	.....	2,400,000	9.00	768
Albany	220,000	8.5	400	47	17,000,000	20.00	77
Experimental sand filters Pittsburgh	670	6.8	45	7	34,000	24.0	51
St. Louis	1,600,000	14.0	750	54	80,000,000	50.0	50
Kansas City	300,000	20.0	1,700	85	15,000,000	72.0	50
Washington	6,690,000	14.0	8,000	570	75,000,000	225.0	11

Basin	Horizontal velocity, mm. per sec.	Mm. in depth of water removed per sec.	Type of basin, Fig. 253	Value of a/t for 75 per cent. removal	Hydraulic value of smallest particles removed*	Diam. of smallest particles removed, mm.
East Jersey	8.0	2.78	Special	2.00	5.56	0.077
Jewell, Pittsburgh	....	0.90	1	3.00	2.70	0.047
Warren, Pittsburgh	....	0.80	2	2.00	1.60	0.034
†Detroit, Mich	43.2	0.65	....	....	0.67	0.020
Ithaca	6.0	0.30	4	1.67	0.50	0.018
Watertown	.0	0.286	4	1.67	0.48	0.018
†Cleveland, O	....	0.368	B	1.07	0.368	0.015
†Baltimore, Md	15.0	0.408	....	....	0.51	0.017
World's Fair, Chicago, 1893 sewage	....	0.366	B	1.00	0.35	0.015
Albany	2.0	0.0363	1½	2.33	0.084	0.007
Experimental sand filters Pittsburgh	0.2	0.0240	1	3.00	0.072	0.007
St. Louis	1.0	0.0235	1	3.00	0.070	0.007
Kansas City	2.0	0.0235	4	1.67	0.039	0.005
Washington	3.0	0.0053	4	1.67	0.009	0.003

* See definition on p. 705. † **Data added by Authors.**

Table 215. Comparison of Different Arrangements of Settling Basins[1c]

Description of basins	Line in Fig. 253	Values of $\frac{a}{t}$: One-half removed	Values of $\frac{a}{t}$: Three-fourths removed	Values of $\frac{a}{t}$: Seven-eighths removed
Theoretical maximum (cannot be reached)	A	0.50	0.75	0.875
Surface skimming, Rochner-Roth system	B	0.54	0.98	1.37
Intermittent basins, reckoned on time of service only	C	0.63	1.26	1.89
Continuous basin, theoretical limit	D	0.69	1.38	2.08
Close approximation to ditto	16	0.71	1.45	2.23
Very well baffled basin	8	0.73	1.52	2.37
Good baffling	4	0.76	1.66	2.75
Two basins tandem	2	0.82	2.00	3.70
One long basin, well controlled	1½	0.90	2.34	4.50
Intermittent basin, in service half time	E	1.26	2.50	3.80
One basin, continuous	1	1.00	3.00	7.00

Values of $\frac{a}{t}$ **necessary to secure the removal of ½, ¾, and ⅞ of the particles of a given size, with basins of different arrangements, and without regard to excessive bottom velocities or the favorable effect of contact with the already subsided particles, are given in Table 214.**

Time. The diameter of particles in millimeters, such that 75 per cent. will be removed with continuance of operation, may be computed, according to Hazen, by the formula:

$$d = 0.0027f\sqrt{\frac{\text{million gallons daily}}{\text{area of basin in acres}}}\sqrt{\frac{60}{t+10}}$$

"in which f is a factor depending upon the arrangement of basins and baffling. Use 1.73 for a basin with one inlet and one outlet well separated; 1.41 for two basins through which the water passes successively; 1.22 for a well-baffled basin or other specially good arrangement. $f = 1.00$ is a theoretical limit not reached in practice. In the last term, t is temperature in degrees Fahrenheit. For comparisons use $t° = 50$ in all cases. The rule does not apply for separations above 0.05 mm. It is not precise, but it affords a convenient basis for comparing various sedimentation and coagulating basins."[1c]

Effect of Temperature. Because of greater viscosity of water at low temperatures, a particle of silt will settle twice as fast at a temperature of 23°C. (74°F.) as at 0°C. (32°F.). The rate of settling at different temperatures varies as $\frac{t°\text{F.} + 10}{60}$. Annual average temperature of water in lakes, ponds, and open reservoirs in Northern United States is about 10°C. = 50°F.

Practical Considerations. Quantities of solids carried in suspension by several American rivers are given in Table 22, p. 63. Results obtained in various subsiding and coagulating basins are given in Table 214. Experiments at New Orleans showed that from Mississippi River water, containing an average of 650 p.p.m. of suspended matter and an average turbidity of 600 p.p.m., the amounts of suspended matter given in Table 216 would be deposited.

Table 216. Estimated Amounts of Suspended Matter Which Would Be Deposited from Mississippi River Water

Period of subsidence, hours	Suspended matter, parts per million	Percentage removed		Mud removed*	
				Per million gallons	
		Silica turbidity	Suspended matter	Cubic yards	Tons
12	215	19	33	2.28	2.48
24	290	28	45	3.07	3.35
48	350	37	54	3.71	4.05
72	385	42	59	4.08	4.45

*Specific gravity of mud at New Orleans = 1.3; weight of mud per cu. ft. = 81 lb.

Results. As a rule, well-baffled basins having a capacity equal to 6 hrs'. flow will remove particles larger than 0.02 mm. in diameter. Basins having a capacity equal to 24 hrs'. flow will remove particles larger than 0.007 mm. At Washington, D. C., in a succession of three reservoirs (see Table 209, p. 633) holding a week's supply, particles larger than 0.003 are removed.

Removal of the bulk of the suspended matter is usually all that is necessary as a preliminary to filtration. Suspended matter in *clay-bearing streams* cannot be completely removed even after weeks of subsidence. Intermittent basins possess no advantage over properly designed continuous basins. Allowing for the time out of service, they do only slightly more efficient work, and furthermore deliver the water at a lower elevation. Ordinary baffling raises the efficiency of continuous above that of intermittent basins. Horizontal basins should have a length not exceeding one and one-half times their width. Greatest efficiency is obtained in a high vertical basin with the entering water well distributed at the bottom and the effluent skimmed from many points at the surface; but this type is impracticable for large plants. Where the entering water is colder than the air, the water in the basin will stratify.

Baffles.* Because efficiency of basins depends in part upon area of bottom surface upon which sediment can be deposited, and because best results are

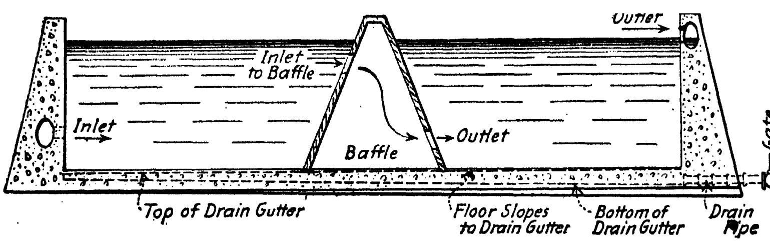


FIG. 254.—Subsiding basin with A-frame baffle.

obtained when the incoming water can be brought into contact with the sediment and kept from mixing with the partially clarified water, it is well to divide horizontal basins into consecutive compartments by baffles. Of these, the *A-frame baffle*, shown in Fig. 254, is the most useful. This baffle receives the water from the upstream compartment through a number of openings near the surface and discharges it near the bottom of the basin through corresponding openings. They are usually built of concrete slabs supported by A-frames of steel or reinforced concrete, or of plank supported by steel frames. Bottom velocities affect the deposition of sediment. Sediment once deposited is not so easily conveyed by the water as before.

Bibliography, Chapter 28. Subsidence

1. *T. A. S. C. E.*, Vol. 53, 1904. **a**, p. 45; **b**, p. 70; **c**, p. 50; **d**, p. 47. **2.** *E. N. R.*, Aug. 30, 1917. p. 417.

* Loss of head in a closely baffled mixing basin was investigated at Greensboro, N. C.; see E. N. R., Mar. 19, 1925, p. 476.

CHAPTER 29

AERATION

Aeration consists in bringing water into intimate contact with air in order to introduce additional oxygen for the oxidation of iron, and for washing out gases and volatile odors. Water sprayed in fine streams from varying heights absorbs oxygen rapidly. For the solubility of oxygen, see p. 583.

The removal of the odor of vegetable matter is difficult. The oxidizing effect upon the organic matter is negligible.

Aeration has successfully removed algal odors from waters at Newark, N. J.; Grassy Sprain reservoir, Yonkers, N. Y.; Whiting St. reservoir, Holyoke, Mass.; Ludlow reservoir, Springfield, Mass. It is undertaken on large scale for New York City's Catskill supply through nozzles causing exposure of water in spray for a few seconds under at least 16-ft. head (see p. 647). Such aeration not only oxygenates, but reduces free carbonic acid from about 20 to to 5 p.p.m. and removes probably three-fourths of odors of growth and decomposition. The Catskill aerators permit draft from reservoir depths where the oxygen is depleted.

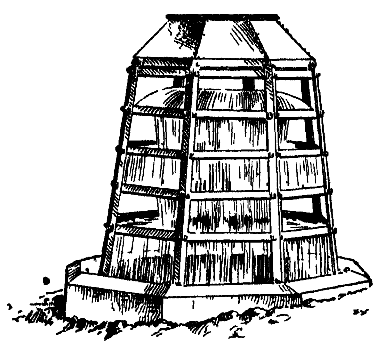


FIG. 255.—Aerator at Winchester, Ky., pumping station. (Wheeler.)

Aerators. Aeration may be accomplished by cascades, fountains, and sprays. Small streams are better than large. Cascades economize head, and, when the water is distributed in thin sheets, are often efficient.

Cascade aerators are built in the form of steps, as at Dallas,[1] and in the form of an inclined plane covered with iron plates on which are placed staggered cast-iron projections arranged to break up the water and bring it into contact with air, as at Norristown.[2] At Indiana University,[3] air was forced into the water; at Oshkosh,[4] multiple-jet inspirators were used. Superimposed pans with alternate perforated and plain sections (Fig. 225), combine the

advantages of spray and cascade aerators.* Other examples are at Danville, Va., Long Beach, N. Y., Oklahoma City, Okla., and the new plant at Providence, R. I. At Oklahoma City, the process is of doubtful value.

Table 217. Oxygen Absorbed by Water Sprayed from Various Heights
Oesten

Height of fall		Oxygen absorbed in parts per million
Centimeters	Inches	
10	4	1.21
25	10	1.79
50	20	2.52
100	40	6.50
200	80	7.33

Table 218. Carbonic Acid Left in Solution after Aeration by Exposure in Drops
G. C. and M. C. Whipple

	Carbonic acid (parts per million)			
At the start	5.0	10.0	25.0	50.0
After 0.5 sec	4.1	6.9	13.8	23.4
After 1 sec	3.5	5.3	9.3	14.0
After 2 sec	3.0	4.1	6.2	8.5
After 5 sec	2.5	3.0	3.8	4.5
After 15 sec	2.1	2.1	2.1	2.1

Table 219. Hydrogen Sulfide Left in Solution after Aeration by Exposure in Drops
G. C. and M. C. Whipple

Time	Sulfureted hydrogen (parts per million)	Odor
At start	15.2	Faint
After 1 sec	10.2	Very faint
After 1.5 sec	5.0	Very faint
After 2.0 sec	2.6	None

Spray-nozzle Aerators. Good dispersion is produced when the water revolves in the pipe before it issues from the nozzle; various spray nozzles on the market embody this principle. Pressure nozzles are the most effective devices for aerating water. They require, however, high heads; are affected by winds; and must be located above pools, readily befouled with debris. The conical nozzle, with floating conical center, developed at Sacramento[5],† and also used at Peabody, Mass.,‡ and elsewhere, is especially adapted to low heads. *Tests* on models by New York Board of Water Supply, 1908 and 1910, showed practicability of casting thin bronze shell with spiral vanes in one piece. Shape finally selected for aerating Catskill water below Ashokan and Kensico dams is made up of cylindrical base surmounted by conical tip. For first installation tip opening was $1\frac{5}{8}$ in. diameter. In base are cast 3 vanes, equally spaced on circumference, and projecting nearly to center of waterway.

* See details of aerator box for South Norwalk, Conn., in *J. N. E. W. W. A.*, March 1916.
† Made by the Water Works Supply Co., Call Bldg., San Francisco.
‡ Plant of American Glue Co.

Angle between vanes and axis of nozzle varies from parallelism at base to 60° at top. Function of vanes is to set up rotary motion in jet, accelerating breaking up of jet into drops. Jet, 15 to 25 ft. high, gives effective aeration. Tests showed that this rotary motion gave jets greater resistance to distortion by wind, as compared to jets from vaneless nozzles. Tests on 1⅜-in. bronze nozzles (Nov., 1910), with long tip and with short tip (also base was tested alone), using heads between 10 and 21 ft.: coefficient for discharge for base only (3 in. diam.) was 0.58; short tip, 0.89 to 0.99 (1⅜ in. diam.); long tip, 0.99 (1⅜ in. diam). Quantity discharged varied from 0.28 to 1.79 cfs. Figure 256 shows nozzle adopted.

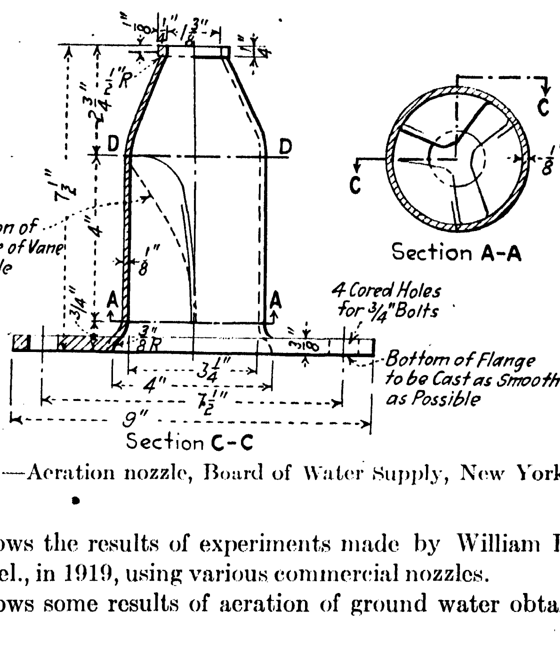


FIG. 256.—Aeration nozzle, Board of Water Supply, New York.

Table 220 shows the results of experiments made by William Easby Jr., at New Castle, Del., in 1919, using various commercial nozzles.

Table 222 shows some results of aeration of ground water obtained from various sources.

Table 220. Removal of CO_2 by Spray Nozzles, Newcastle, Del.

Style of nozzle and diameter of orifice	Carbon dioxide, parts per million		Temperature—Fahr. Water		Temperature—Fahr. Air		Pressure at base of nozzle, pounds	Discharge		Height of nozzle tip above ground, feet
	Before aeration	After aeration	Before aeration	After aeration	Before aeration	After aeration		Gallons per min.	Gallons per day	
Spraco ½″	77.0	6.6	58	66	70	70	5	12.2	17,580	5.9
Spraco ½″	84.5	5.5	58	66	68	68	5	12.2	17,580	5.9
Spraco ½″	74.4	3.1	62	65	73	73	5	12.2	17,580	1.9
Spraco ½″	74.4	3.5	62	65	73	74	5	12.2	17,580	1.9
Spraco ⅝″	84.5	6.4	58	66	68	68	5	20.6	29,670	5.9
Spraco ¾″	74.8	3.1	61	64	72	72	5	33.5	48,240	5.9
Priestman ¾″	73.9	7.5	60	64	71	71	5	.....	......	5.9
Worcester 13/16″	79.2	5.1	58	64	72	72	5	29.5	40,250	5.9
Worcester 13/16″	79.2	8.4	.....	.....	.....	.....	5	29.5	40,250	1.9
Worcester 13/16″	74.6	7.3	.....	.....	.....	.....	5	29.5	40,250	1.9
Worcester 13/16″	74.6	8.6	.....	.....	.....	.....	5	29.5	40,250	1.9

Table 221. Typical Spray Aerators

Manual of American Water Works Practice 1925, p. 193

Locality	Date installed	Nominal capacity Mgd	No nozzles	Size orifice inches	Rating g p m at 10 lbs	Floor space sq ft: Total	Floor space sq ft: Per Mgd	Ave nozzle head ft	Exposed or enclosed	Gravity or pumped	Source of information
New York N Y											
Kensico	1913	400	1600	$1\frac{3}{8}$	170	77 000	193	20	Exp	G	Fred F Moore & Frank Hale
Ashokan	1913	400	1600	$1\frac{3}{8}$	170	77 000	193		Exp	G	
Richmond Va	1924	30	300	$1\frac{1}{16}$	100	53 000	1770	11	Exp	I	F O Baldwin
Ft Worth Tex	1918	14	64	$1\frac{3}{8}$	150	9 43[illegible]	67	17	Exp	G	W S Mahlie
Whiting Ind	1920	4	22	$1\frac{5}{16}$	150	1 440	360	10	Louvre	P	Paul Hansen & Bernard Jeup
Warsaw Ind	1924	2	12	$1\frac{5}{16}$	140	1 06[illegible]	[illegible]33	9	Louvre	P	Paul Hansen
Providence R I											
Influent	1925	100	373	$1\frac{3}{8}$	17[illegible]	2[illegible] 700	277	27	Exp	G	Malcolm Pirnie
Effluent	1925	60	800	1 2	41	10 000	167	[illegible]	Exp	G	Malcolm Pirnie
Danville Va	1924	5	42	$1\frac{1}{4}$	76	1 600	320	14	Exp	I	Malcolm Pirnie & Richard Messer
W Palm Beach Fla	1921	8	50	$1\frac{1}{8}$	94			23	Exp	I	Malcolm Pirnie
			50	1 [illegible]	77						

Note—Information in response to a questionnaire by committee of A W W A

Contact. Interchange of gases may be accelerated by contact, as accomplished by a tower or trickler filled with coke slag, or stone (several European plants use brick, also wooden slats), such as used in deferrization plants The

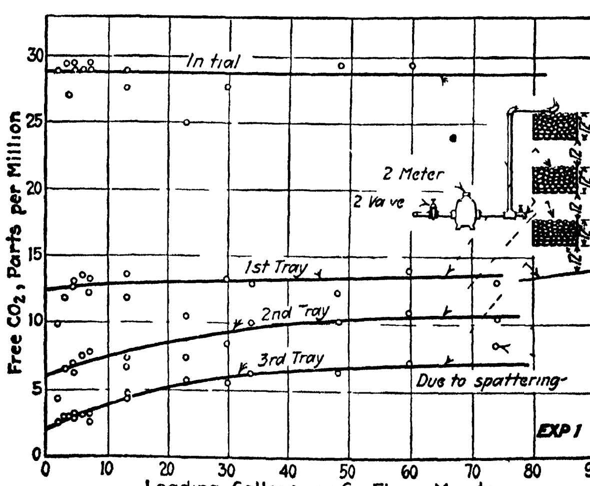


FIG 257—Experiments with multiple coke trays [7]

contact principle applies in all filters, especially where waters containing heavy growths of algæ, much organic matter, or colloidal iron, make such treatment necessary Oxygen is condensed in the films on the sand during the idle period and given off during the working period [6]* W Donaldson,[7] who experimented with various aerating devices, recommended multiple

* See Ludlow Filter at Springfield Mass, *J N E W W A*, Vol 21, 1907, p 279

coke trays, filled with coke, for the Memphis deferrization plant. The results of his experiments with this type of aerator are shown by Fig. 257.

Table 222 shows the results obtained by aerating ground water to remove carbon dioxide.

Table 222. Some Results of Aeration of Ground Water*

Locality	Type of aerator	Carbon dioxide, p.p.m. Before	After
Merrimac, N. H.	Sprays and 10′ stone bed	29.6	5.8
Brookline, Mass.	Sprays and 10′ coke bed	24	8
Lowell, Mass.	2″ riser pipes & 10′ coke bed	23	13
Middleboro, Mass.	Spray above 10′ coke bed	41	4
Peabody, Mass., (A. G. Co.)	Sacramento nozzles and one coke tray	30	6
Far Rockaway, N. Y.	Single large riser pipe	30	12
Long Beach, N. Y.	Spray nozzles	38	11
Rockaway Park, N. Y.	11 wooden steps	55	31
Virginia Beach, Va.	Spray nozzles	125	20
Le Roy, O.	Spray nozzles	64	20
Shelby, O.	Stack of 3 coke trays	48	16
Wadsworth, O.	Spray nozzles	35	6
Xenia, O.	Stack of 3 coke trays	30	10
Memphis, Tenn†	Air lift wells	120	32
	Stack of 4 coke trays	32	7.5
Superior, Wis.	Stack of 4 perforated pans	12	4
Excelsior Springs, Mo.	Multiple spilling pans	73	30

* From various sources.
† See footnote, p. 255.

Relation to Filtration. Pirnie[8] and Bunker[9] have shown that aeration to remove carbon dioxide, following the application of sulfate of alumina to coagulate water, not only promotes coagulation, but the efficiency of filtration which follows. At Watertown, this practice also prevented "red-water" troubles, and decreased the dose of chlorine required to sterilize the water. It also minimized the odors due to sulfite waste in the polluted raw water. At West Palm Beach, Fla., the results obtained with and without aeration show its advantages.

Table 223. West Palm Beach, Fla. Results of Aeration of Water Treated with 65 p.p.m. of Sulfate of Alumina
Pirnie

Source of sample	Parts per million: Color	Alkalinity	Carbon dioxide	pH value
Lake	148	36	3	7.2
Dosed water			30	5.6
Aerated spray			16	5.8
Filter influent			8	5.3
Filter effluent	8	3	6	5.2

Bibliography, Chapter 29. Aeration

1. *E. N. R.*, June 14, 1923, p. 1044. 2. *E. N.*, May 6, 1915, p. 853. 3. *Proc. Ind. Acad. Sci.*, 1919, p. 71. 4. *E. N.*, Vol. 78, 1917, p. 237. 5. *E. N. R.*, Sept. 7, 1922, p. 384. 6. Dunbar: *Zeit. f. Hygiene*, Vol. 22, 1896, p. 68. 7. *E. N. R.*, May 17, 1923, p. 874. 8. *Munic. County Eng.*, Vol. 67, 1924, p. 22. 9. *E. N.*, Feb. 17, 1916, p. 334.

CHAPTER 30

COAGULATION AND DISINFECTION

COAGULATION

Coagulation,* as applied to water, is the application of certain harmless chemicals in minute quantities to produce a precipitate that includes the mud, clay, organic matter, and bacteria, and combines them into aggregates (flocs) of a size easily removable either by subsidence alone or by subsidence and filtration. Usually water containing more than 30 p.p.m. of turbidity or more than 20 p.p.m. of color, must be coagulated before filtration. Coagulation serves two purposes: (*a*) it gathers into groups or flocs, particles which in a dispersed condition could not be removed by subsidence or filtration; (*b*) it forms sponge-like layers or films in the filter sand which allow water to pass at a high rate, while at the same time retaining bacteria and suspended matter. These two functions are called surface adsorption and straining. The vast surface presented by the jelly-like compound which is formed by the addition of a coagulant to an alkaline water causes the adsorption of color, organic matter, and other substances, which exist in true or colloidal solution. In slow filters the bacterial films accumulated on the sand particles perform the same function.

Hydrogen-ion Concentration. Any acid (or alkali) when dissolved in water tends to dissociate into the hydrogen ion and the other component ions. This splitting into ions is known as "ionization." All acids (and alkalies) do not ionize to the same degree. For example, hydrochloric acid (HCl) ionizes about 90 per cent., but acetic acid (HAc) ionizes only about 2 per cent., and boric acid (H_3BO_3) about 0.005 per cent. An acid which ionizes freely is called a strong acid; others weak acids. The properties† of acids and alkalies vary in intensity with the degree of ionization. In other words, the aggressiveness or potentiality of an acid does not depend upon the total quantity of acid per unit volume (its concentration), but upon the total quantity of ionized hydrogen per unit volume. Alkalies and all salts ionize in a similar manner, and their aggressiveness depends also upon the degree of the ionization, known as the "hydrogen-ion concentration," usually expressed as the pH value, which is the logarithm of the reciprocal of the weight in grams of hydrogen-ions present in 1 liter. (See Table 225, p. 653.)

In any solution, the product of the hydrogen and other ions is constant. Pure water is neutral, that is, neither acid nor alkaline, and its hydrogen-ion concentration is $pH = 7$. A pH value of above 7 indicates an excess of OH (Hydroxyl) ions or a potential alkalinity; below 7, an excess of H ions, acidity.

* Technically, the change from a dispersed phase into a system made up of solid masses and liquid.

† Taste, action on skin, reaction with indicators, etc.

The simplest method of determining the *pH* value of any solution (*e.g.*, water), is the colorimetric method.* Certain substances known as buffer salts are utilized in preparing indicator solutions of definite *pH* values with which the sample of water similarly treated is compared. Table 224 lists some of the indicators used, with their *pH* range and color change. Other methods of determining *pH* are also being used† and are being refined.

Importance of the *pH*-Value. The formation by coagulation of a good floc‡ in as short a time as possible depends to a great extent on the *pH* value of the treated water. It is important that the coagulant be precipitated as completely as possible,§ and recent research indicates that there is an optimum *pH* value for each water and coagulant at which the maximum amount of coagulant is thrown out of solution, but that above or below this point precipitation is not complete. This optimum point has been found to be anywhere from $pH = 5$ to $pH = 7$. The optimum point should be determined for each water which should then be brought to the correct *pH*‖ by proper chemical treatment.¶ See Tables 224 and 225.

Theory of Colloids. Colloids are simply matter in a finely divided or highly dispersed condition, yet not so finely divided as to form true molecular solutions. In order to be dispersed in colloidal form, a substance must be so finely divided that it will remain in suspension indefinitely. A lump of clay-schist sinks rapidly, but when disintegrated by water the clay particles of which it is composed remain in suspension indefinitely, like those in the Mississippi River water. Fine particles do not follow the law of coarse subsiding particles. They possess molecular movements (called Brownian) which are negligible for coarse particles, and remain suspended indefinitely. Particles of vapor and dust in the air are often in the colloidal state. Gold, paper, clay, iron, and other substances may be so finely divided as to remain suspended in pure water indefinitely.

The force which holds these very fine particles in suspension is a property of their surfaces, a film produced by the selective powers of adsorption which these surfaces possess, *e.g.*, the condensation or orientation of oxygen molecules on the surfaces of the sand grains in a filter; or the electrical charge which causes the unprecipitated particles of water in the atmosphere to repel one another, thus preventing their coalescence and consequent precipitation.

Particles in the colloidal state are smaller than 0.0001 mm. in diameter and in some cases no larger than 0.000,006 mm. The enormous increase in surface area due to division of matter, greatly augments the capacity of the

* See Clark & Lubs, "Colorimetric Determination of *pH*," *J. Bact.* Vol. 2, 1917, pp. 1, 109, 191. Clark, "Determination of H-Ions," Williams-Wilkins Co., 1924.

† Such as the electric method. See Clark's "Determination of Hydrogen Ions," 2nd Edition 1924. Also "Standard Methods of Water Analysis," *A. P. H. A.*, 1925.

‡ See Reprint 813, *U. S. Pub. Health Reports*, "An Experimental Study of Relation of *pH* to the Formation of Floc in Alum Solutions," by Theriault and Clark. 1923.

Massink & Heymann, "Significance of *pH* in Drinking Water," *J. A. W. W. A.*, Vol. 8, 1921, p. 239.

§ Wolman & Hannan, "Residual Al. Compounds in Filter Effluents," *Chem. & Met. Eng.*, Vol. 24, Apr. 27, 1921.

Hatfield, "Hydrogen-ion Concentration and Sol. Al. in Filter Effluents," *J. A. W. W. A.*, Vol. 11, 1924, p. 554.

Catlett, "Optimum *pH* for Coag. of Various Waters," *J. A. W. W. A.*, Vol. 11, 1924, p. 887.

‖ Baylis, "Use of Acids with Alum in Water Purification and Importance of *pH*," *J. A. W. W. A.*, Vol. 10, 1923, p. 365.

Norcom, "Purification of Colored Water at Wilmington, N. C.," *J. A. W. W. A.*, Vol. 11, 1924, p. 96.

¶ At Washington N. C., deep well water high in alkalinity was mixed with the water being treated to raise the *pH*.

substance for adsorption. This has an important bearing upon the power of aluminum and other hydrates used as water coagulants which, when precipitated in water, pass gradually through the colloidal or dispersed state into the solid state and in doing so offer, because of their small size, a correspondingly vast surface for the adsorption* of suspended matter such as clay, color, and bacteria.

Colloidal solutions are of two kinds, those wherein the substances are dispersed in water (where water is the external phase) and those wherein water is within the expanded spongy structure of the substance (where water is the internal phase). Colloidal gold is an example of the first, gelatin of the second kind of solution. Where water is the external phase, and where the colloids are suspended electrically, the particles will migrate or move when an electric current is present, to the positive or negative poles, depending upon the preferential adsorption of negative or positive electrical charges by the particles. Colloidal clay has a negative, aluminum hydrate a positive, charge. Colloids of like charges repel, while those of opposite charges attract one another. In Posen, Poland, the colloidal coloring matter in the deep-well water and the colloidal iron in the aerated shallow-well water mutually precipitate each other when the waters are mixed in the right proportions. Where water is the internal phase, some of the gelatin-like substances may pass in and out of solution by the expansion and contraction of the honeycomb-like structure of the substance. These colloids are "reversible." The existence of this structure may explain how certain vegetable matters, like vegetable albumens, may retard coagulation by preventing contact of the particles of coagulant; *i.e.*, they interpose their structure between the particles. Other colloids which, when thrown out of solution, cannot be taken up again without redivision are called "irreversible."

Electrolytes (dissolved salts which conduct current) tend to precipitate colloids. This is why coagulation in soft waters is more difficult than in hard. Any contact surface, particularly one covered with a film of already coagulated substance, favors coagulation; likewise agitation, mixing, friction, time, or high temperature, all of which tend to bring the separated particles into contact.

Table 224. Indicators Used in Determination of pH Value

Indicator	pH Range	Color change	
		Acid	Alkaline
Thymol blue	1 2–2 8	Red	Yellow
Brom-phenol blue	3 0–4 6	Yellow	Blue
Brom-cresol purple	4 0–5 6	Yellow	Blue-green
Brom-cresol purple	5 2–6 8	Yellow	Purple
Brom-thymol blue	6 0–7 6	Yellow	Blue
Phenol red	6 8–8 4	Yellow	Red
Cresol red	7.2–8 8	Yellow	Red
Thymol blue	8 0–9 6	Yellow	Blue
Cresol phthalein	8 2–9 8	Colorless	Red

* This adsorption is due to the holding of the molecules of one compound against the surface of another compound. Probably the zone of influence, where the molecules are oriented, is only one molecule thick.

Table 225. Relation between pH Value in Terms of Normality and the Concentration of Ionized Hydrogen in Completely Ionized Solutions of Acids or Alkalies

Reaction	pH	Normality	Grams of H per Liter
Alkaline	14.0	N/1 (Alkali)	0.00000000000001
Alkaline	13.0	N/10	0.0000000000001
Alkaline	12.0	N/100	0.000000000001
Alkaline	11.0	N/1000	0.00000000001
Alkaline	10.0	N/10000	0.0000000001
Alkaline	9.0	N/100000	0.000000001
Alkaline	8.0	N/1000000	0.00000001
Neutral	7.0	N/10000000 (Neutral)	0.0000001
Acid	6.0	N/1000000	0.000001
Acid	5.0	N/100000	0.00001
Acid	4.0	N/10000	0.0001
Acid	3.0	N/1000	0.001
Acid	2.0	N/100	0.01
Acid	1.0	N/10	0.1
Acid	0.0	N/1 (Acid)	1.0

The anions* present may determine the *pH* range over which the coagulation takes place; *e. g.*, Miller[1] has shown that the SO_4 ion produces a comparatively good floc over a wide range of *pH* values.

Coagulants. The coagulating chemical must be one which will be cheap, form an insoluble, flocculent precipitate which will have a large surface, and yet settle readily. The chemicals most commonly used are compounds of aluminum and iron.

Sulfate of alumina, frequently though erroneously called alum, is the most commonly used coagulant. It is usually basic; *i.e.*, it contains more hydrate than will combine with sulfuric acid to form a neutral salt. Best sulfate of alumina contains 17.0 to 18.0 per cent. of Al_2O_3—which should be in excess of the amount theoretically required for the sulfuric acid and not more than 0.75 per cent. of iron oxide (Fe_2O_3). It should be furnished in lumps 0.75 in. to 3.0 in. in diameter for making solutions, or ground so that not less than 95 per cent. shall pass a woven sieve having 10 meshes per lin. in. and 100 per cent. shall pass a sieve having 4 meshes per in., for feeding dry.†

For treating certain waters sulfates which are only slightly basic are best. Acid sulfates are difficult to ship and use. Where a low *pH* value is demanded for coagulation, it is better to add acid separately.

In 1915, C. P. Hoover of Columbus, Ohio, began making crude sulfate of alumina by dissolving bauxite in sulfuric acid, using a pug-mill for completing the reaction and afterward discharging and drying the product upon a concrete floor. This chemical has displaced the commercial product in many plants. The objection to its use is the relatively large amount of inert matter which it contains and whose source is the natural bauxite. In many plants, especially the larger ones, it has effected marked economies. The bauxite used should contain not less than 54 per cent. Al_2O_3; not over 5 per cent. Fe_2O_3; and not over 3 per cent. SiO_2. The commercial sulfuric acid used

* A solution dissolved by electrolysis into ions and anions, positive and negative, respectively; $H_2SO_4 = H_2 + SO_4$.

† For all chemicals for treating water, use specifications prepared by American Waterworks Association. See Manual of American Waterworks Practice, 1925, p. 707.

should have a specific gravity of about 1.706 (60°Bé.) and not less than 1.648 (57° Bé.)[2]* Crystallization may be avoided by feeding solution.

The theoretical maximum alkalinity† computed as $CaCO_3$ that can react with 1 g.p.g. (grain per gallon) of ordinary 17.5 per cent. sulfate is 8.2 p.p.m., or 1 p.p.m. of alkalinity to 2.08 p.p.m. of sulfate. In practice, reduction in alkalinity may be as low as 60 per cent. of theoretical or 5.0 p.p.m. to 1 g.p.g. of sulfate. CO_2 set free by the reaction with the alkali in water averages 3.4 p.p.m. to 1 g.p.g. (1 g.p.g. = 17.12 p.p.m.).

Colloidal clay absorbs some added coagulant and prevents it from reducing the computed amount of alkali. As color and turbidity decrease and as alkalinity and time increase, the theoretical reduction of alkalinity is approached and finally all the sulfate is replaced by carbonate, which latter hydrolyzes with the production of aluminum hydrate, $Al_2(OH)_6$. The chemical reaction is illustrated as follows:

$$Al_2(SO_4)_3 + 3CaCO_3, H_2CO_3 = Al_2(OH)_6 + 3CaSO_4 + 6CO_2$$

The reaction between sulfate of alumina and water is a time reaction and is retarded by cold.

Many *very highly colored waters*, like that of the Great Swamp, in the South[3] which have a carbonate alkalinity of 9, a low *pH* value, and a color of 150 p.p.m., are often really acid with vegetable acids.‡ Addition to this water of up to 1.5 g.p.g. of sulfate of alumina produced no apparent reaction in 24 hr. A further addition, up to 2.5 g.p.g., removed a large proportion of color, but addition of more than 4 g.p.g. could not reduce color below 25 p.p.m. To decolorize such waters completely, either the bulk of the precipitate must be increased by addition of more sulfate with alkali to react with it, or by the addition of chlorine. An excess of sulfate of alumina lightens the color of natural water; an excess of alkali deepens it. The amount of color which can be removed by the same dose of sulfate of alumina varies greatly with the character of the coloring matter. (See also H-ion Concentration, p. 651.)

Chlorine. Sometimes compounds of iron and coloring matter exist in water; such cannot be completely decolorized by sulfate of alumina alone. In some cases the greater part of this residual color may be removed by treating the water with Cl before adding the sulfate. At Exeter, N. H., in 1914, the addition of an average of 0.5 p.p.m. of chlorine to the raw water with an average of 32.8 p.p.m. of sulfate of alumina, reduced the color from an average of 56 to an average of 6.6 p.p.m. During the previous year, without the use of chlorine, color could not be reduced below 27 parts, even with an increased dose of sulfate of alumina.§ This experience has been repeated at Belfast, Me., where an impounded reservoir water is treated, and at the Arlington Mills, Lawrence, Mass., where the Spicket River, which has had a color as high as 180 p.p.m., is treated; also at Louisville, Ky., Avalon, Md., Newport News, Va., Virginia Beach, Va., Davenport, Ia., London, Eng., and elsewhere.

* Natural aluminum hydrate or hydrated oxide.

† Alkalinity: the dissolved carbonates and bicarbonates of calcium, magnesium, sodium, etc.

‡ Some vegetable acids are weak and react alkaline with litmus, methyl orange, erythrosine, and other indicators of relatively high acidity; they react acid to phenolphthalein and other indicators of low acidity.

§ See also Gammage, *E. N. R.* Sept. 7, 1922, p. 391; Weston, "Use of Chlorine to Assist Coagulation," *J. A. W. W. A.*, Vol. 11, 1924, p. 446; Howard, "Modified Pre-chlorination Treatment," *Can. Eng.*, Feb. 26, 1924, p. 283.

In Belfast, during the winter, it is impossible to reduce the color sufficiently or even to coagulate the water completely without the addition of chlorine. At the Arlington Mills, the color was reduced during June, 1915, from an average of 83 p.p.m. to an average of 10, the dose of chemicals being 62 p.p.m. of sulfate of alumina, 10 of soda, and 0.1 part of chlorine. Without chlorine, reduction of color to 10 p.p.m. or less was impossible. At certain other places the use of chlorine to aid coagulation is of no advantage.

Ferrous sulfate in conjunction with lime is a good coagulant for turbid, alkaline waters such as those of the Missouri and Ohio basins. Its formula is $FeSO_4, 7H_2O$. The best form, known as "sugar of iron" or "sugar sulfate," is partially dehydrated and contains over "100 per cent." of sulfate calculated as $FeSO_4, 7H_2O$ and less than 1 per cent. of foreign matter. Ferrous sulfate forms a coagulum of greater specific gravity and is considerably cheaper than sulfate of alumina. On the other hand, it must be used in conjunction with lime. The addition of two chemicals requires greater, and more skilful, supervision than the use of one. But the addition of lime where temporary hardness is high softens the water. Ferrous sulfate and lime cannot be well used with waters which are high in color or which are alternately turbid and colored. Neither can they be well used with soft waters, because any surplus lime would give the water a caustic alkalinity.

The reaction is complicated, but may be given as follows: The sulfate reacts with the bicarbonates in the water forming ferrous bicarbonate, *viz:* $FeSO_4 + CaCO_3H_2CO_3 = FeCO_3H_2CO_3 + CaSO_4$. This would oxidize and precipitate too slowly for any practical use. Lime must be added to absorb the CO_2 and complete the reaction,

$$FeCO_3H_2CO_3 + Ca(OH)_2 = Fe(OH)_2 + CaCO_3H_2CO_3.$$

The $Fe(OH)_2$ formed is a gelatinous precipitate similar in action to that produced by sulfate of alumina. It is, however, slightly soluble, but by oxidation is changed into the insoluble ferric hydrate, $Fe_2(OH)_6$, which is the coagulant desired. A slight excess of dissolved oxygen is needed to convert ferrous to ferric hydrate. Iron bacteria are apt to grow in sulfate solutions, even those containing from 2000 to 4000 p.p.m. of Fe. $Fe_2(OH)_6$ can be precipitated directly by the addition of ferric sulfate* ($Fe_2(SO_4)_6 + 9H_2O$), but the chemical has been too costly and does not react with coloring matter so well as the basic sulfate of alumina. Sulfate of alumina may contain a small amount of ferric sulfate ("alumino-ferric") without reducing the efficiency of the coagulant for moderately hard turbid waters.

Barium carbonate is used in removing non-carbonate hardness from water. It is marketed in the form of a white powder (see p. 685).

Sodium chloride, common salt, is used to regenerate Zeolyte (see p. 686). The chemical should contain less than 0.5 per cent. of suspended matter.

Lime† is used for water softening; for an alkali with ferrous sulfate and sulfate of alumina; and to neutralize acid waters. Pure lime has the formula CaO, but ordinary building lime may contain as little as 75 per cent. For water softening, the standard lime should contain 88 per cent. of soluble CaO, and preferably more than 90 per cent. The presence of calcium carbon-

* Sold to dyers as "Nitrate of Iron."
† For volumes and weights, see p. 677.

ate, $CaCO_3$, is not disadvantageous for neutralizing acid waters. Lime should be fresh and crushed or ground into lumps smaller than 2 in. diam. The so-called "pea lime" is being used. It should be stored in tight bins to prevent air-slacking.

Hydrated lime is quicklime which has been slaked by steam. It is more convenient to use than quicklime, especially for dry feeding. It does not require slaking, and is usually purer than most quicklimes. Hydrated lime costs more than quicklime, largely because of its increased weight (32 per cent.) due to hydration, and consequent increased freight charges.

Soda ash,* anhydrous sodium carbonate (Na_2CO_3), containing not less than 58 per cent. Na_2O or 98 per cent. of Na_2CO_3, is used to furnish alkali to waters which are deficient therein and to soften water having "permanent" or "mineral acid" hardness. It should be furnished in powdered form containing no large lumps or crystals, and not more than 0.5 per cent. of matter insoluble in distilled water. Theoretically, 1 part of sulfate of alumina requires 0.5 part of sodium carbonate to precipitate it. With soft, colored waters, only about one-half of the theoretical amount should be added lest the color be redissolved. If double the theoretical amount be added to hard waters, no free CO_2 is formed. Soda ash is the best alkali to use in small plants. It is more convenient than lime and requires a shorter period of coagulation. Its cost is usually more than three times that of lime, for equal weights.

Sodium silicate is used to protect metals against corrosion. See p. 661.

Table 226. Average Quantities of Sulfate of Alumina (17.5 Per Cent.) Required to Coagulate Waters of Various Turbidities†

Turbidity, p.p.m.	Basic sulfate of alumina : 17.5% Al_2O_3 required								
	Parts per 1,000,000			Grs. per gal.			Lbs. per 1,000,000 gal.		
	Av.	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.
Under 10	10	17	5	0.60	1.00	0.30	86	143	43
15	14	20	8	0.83	1.19	0.48	119	170	69
20	17	22	11	1.00	1.30	0.61	143	186	87
40	19	25	13	1.12	1.47	0.74	160	210	106
60	21	28	14	1.21	1.61	0.83	173	230	119
80	22	30	15	1.29	1.74	0.90	184	249	129
100	24	32	16	1.39	1.87	0.96	199	267	137
120	25	34	17	1.45	2.00	1.01	207	286	144
150	27	37	18	1.59	2.18	1.07	227	311	153
200	30	42	19	1.76	2.47	1.14	251	353	163
250	33	47	20	1.92	2.74	1.19	274	391	170
300	36	51	21	2.08	2.99	1.23	297	427	176
400	39	62	22	2.25	3.60	1.28	321	514	183
500	42	70	23	2.45	4.08	1.33	350	583	190
600	47	77	24	2.75	4.50	1.37	393	643	196
700	50	——	24	2.92	——	1.41	417	——	201
800	53	——	25	3.06	——	1.45	437	——	207
900	55	——	26	3.19	——	1.49	456	——	213
1000	56	——	26	3.28	——	1.52	469	——	217

* For volumes and weights, see p. 677.

† Considerably less coagulant is required for some waters if slow stirring or mixing follows addition.

Quantities of Coagulant. The quantities of *sulfate of alumina* required vary greatly with the water. Silt and colloidal coloring matter on the verge of precipitation, require but little coagulant. Freshly dissolved vegetable coloring matter and colloidal clay require much greater amounts. The turbid and slightly colored waters of South Atlantic seaboard are examples of the first class; the waters of swamps and the Mississippi River at New Orleans are examples of the second. Contaminated waters of low turbidity and color require, for the removal of bacteria, a larger dose of sulfate of alumina than necessary to coagulate the water. When waters of high color or turbidity are sufficiently decolorized or clarified, removal of bacteria is adequate. It is best to determine the required quantities of chemicals by means of a number of laboratory experiments with samples of the water to which various amounts of coagulant have been added, the results noted and the best com-

Table 227. Average Quantities of Sulfate of Alumina (17.5 Per Cent.) Required to Coagulate Waters of Various Colors

Color, p.p.m.	Average sulfate of alumina required		
	Parts per 1,000,000	Grs. per gal.	Lbs. per 1,000,000 gal.
10	13	0.76	108
20	16	0.93	133
40	26	1.52	217
60	33	1.93	276
80	40	2.34	334
100	44	2.57	367
120	48	2.80	400
150	54	3.15	450
200	64	3.74	534

Table 228. Average Quantities of Ferrous Sulfate and Lime Required for Various Turbidities

Turbidity of settled water	Cincinnati (1913–14)				New Orleans (1913–14)			
	Ferrous sulfate		Lime		Ferrous sulfate		Lime	
P.p.m.	P.p.m.	Lbs., mg.	P.p.m.	Lbs., mg.	P.p.m.	Lbs., mg.	P.p.m.	Lbs., mg.
50	30	247	14	120	......	......	......	......
100	35	289	16	138	0.3	3	56	471
150	37	310	17	145	0.9	7	65	543
200	40	333	18	151	1.2	10	70	582
250	42	353	19	160	1.5	13	74	617
300	......	......	......	......	1.9	16	77	640
400	......	......	......	......	2.7	23	82	683
500	......	......	......	......	3.6	30	85	709
600	......	......	......	......	4.3	36	88	732
700	......	......	......	......	5.0	42	90	748
800	......	......	......	......	5.8	48	91	760
900	......	......	......	......	6.5	54	92	772
1000	......	......	......	......	7.4	61	94	783
1200	......	......	......	......	8.9	74	97	808
1400	......	......	......	......	10.4	87	100	832
1600	......	......	......	......	11.8	98	102	852
1800	......	......	......	......	13.4	111	105	877
2000	......	......	......	......	14.9	124	108	898

Table 229. Costs of Chemicals Used as Coagulant per 1,000,000 Gals. of Treated Water

P.p.m.	Grs. per gal.	Price of chemical per 100 lbs.																			Pounds per million gals.
		$0.75	$0.80	$0.85	$0.90	$0.95	$1.00	$1.05	$1.10	$1.15	$1.20	$1.25	$1.30	$1.35	$1.40	$1.45	$1.50	$1.55	$1.60	$1.70	$1.75	
1	0.06	0.062	0.066	0.070	0.075	0.079	0.083	0.087	0.091	0.095	0.100	0.104	0.108	0.112	0.116	0.120	0.124	0.129	0.133	0.141	0.145	8.3
5	0.29	0.313	0.334	0.355	0.375	0.396	0.417	0.438	0.459	0.480	0.501	0.522	0.542	0.563	0.584	0.605	0.626	0.647	0.668	0.709	0.730	42.0
10	0.59	0.626	0.668	0.709	0.751	0.793	0.834	0.876	0.918	0.960	1.00	1.04	1.08	1.13	1.17	1.21	1.25	1.29	1.34	1.42	1.46	83.0
15	0.88	0.939	1.00	1.06	1.13	1.19	1.25	1.31	1.37	1.44	1.50	1.56	1.63	1.69	1.75	1.81	1.88	1.94	2.00	2.13	2.19	125.0
20	1.17	1.25	1.34	1.42	1.50	1.59	1.67	1.75	1.84	1.92	2.00	2.09	2.17	2.25	2.34	2.42	2.50	2.59	2.67	2.84	2.92	167.0
25	1.46	1.56	1.67	1.77	1.88	1.98	2.09	2.19	2.29	2.40	2.50	2.61	2.71	2.82	2.92	3.02	3.13	3.23	3.34	3.55	3.65	209.0
30	1.75	1.88	2.00	2.13	2.25	2.38	2.50	2.63	2.75	2.88	3.00	3.13	3.25	3.38	3.50	3.63	3.75	3.88	4.00	4.26	4.38	250.0
35	2.05	2.19	2.34	2.48	2.63	2.77	2.92	3.07	3.21	3.36	3.50	3.65	3.80	3.94	4.09	4.23	4.38	4.53	4.67	4.96	5.11	292.0
40	2.34	2.50	2.67	2.84	3.00	3.17	3.34	3.50	3.67	3.84	4.01	4.17	4.34	4.51	4.67	4.84	5.01	5.17	5.34	5.67	5.84	334.0
45	2.63	2.62	3.00	3.10	3.38	3.57	3.75	3.94	4.13	4.32	4.51	4.69	4.88	5.07	5.26	5.44	5.63	5.82	6.00	6.38	6.57	375.0
50	2.92	3.13	3.34	3.55	3.75	3.97	4.17	4.38	4.59	4.80	5.01	5.22	5.43	5.63	5.84	6.05	6.26	6.47	6.68	7.09	7.30	417.0
55	3.21	3.44	3.67	3.90	4.13	4.36	4.59	4.82	5.05	5.28	5.51	5.74	5.97	6.20	6.42	6.65	6.88	7.11	7.34	7.80	8.03	459.0
60	3.50	3.75	4.01	4.26	4.51	4.76	5.01	5.26	5.51	5.76	6.01	6.26	6.51	6.76	7.01	7.26	7.51	7.76	8.01	8.51	8.76	501.0
65	3.80	4.07	4.34	4.61	4.88	5.15	5.42	5.69	5.97	6.24	6.51	6.78	7.05	7.32	7.59	7.86	8.14	8.41	8.68	9.22	9.49	542.0
70	4.09	4.38	4.67	4.96	5.26	5.55	5.84	6.13	6.42	6.72	7.01	7.30	7.59	7.89	8.18	8.47	8.76	9.05	9.35	9.93	10.22	584.0
75	4.38	4.69	5.01	5.32	5.63	5.94	6.26	6.57	6.88	7.20	7.51	7.82	8.14	8.45	8.76	9.07	9.39	9.70	10.01	10.64	10.95	626.0
80	4.67	5.01	5.34	5.67	6.01	6.34	6.68	7.01	7.34	7.68	8.01	8.34	8.67	9.01	9.35	9.68	10.01	10.35	10.68	11.35	11.68	667.0
85	4.97	5.32	5.67	6.03	6.38	6.74	7.09	7.45	7.80	8.16	8.51	8.87	9.22	9.57	9.93	10.28	10.64	10.99	11.35	12.06	12.11	709.0
90	5.26	5.63	6.01	6.38	6.76	7.13	7.51	7.89	8.26	8.64	9.01	9.39	9.76	10.14	10.51	10.89	11.26	11.64	12.02	12.77	13.14	751.0
95	5.55	5.95	6.34	6.74	7.13	7.53	7.93	8.32	8.72	9.12	9.51	9.91	10.30	10.70	11.10	11.49	11.89	12.29	12.68	13.47	13.87	793.0
100	5.84	6.26	6.68	7.09	7.51	7.93	8.34	8.76	9.18	9.60	10.01	10.43	10.85	11.26	11.68	12.10	12.52	12.93	13.35	14.18	14.60	834.0
110	6.43	6.88	7.34	7.80	8.26	8.72	9.18	9.64	10.10	10.56	11.01	11.47	11.93	12.39	12.85	13.31	13.77	14.23	14.69	15.60	16.06	918.0
120	7.01	7.51	8.01	8.51	9.01	9.51	10.01	10.51	11.01	11.51	12.02	12.52	13.02	13.52	14.02	14.52	15.02	15.52	16.02	17.02	17.52	1001.0
130	7.59	8.14	8.68	9.22	9.76	10.30	10.85	11.39	11.93	12.47	13.02	13.56	14.10	14.64	15.19	15.73	16.27	16.81	17.36	18.44	18.98	1085.0
140	8.18	8.76	9.35	9.93	10.51	11.10	11.68	12.27	12.85	13.43	14.02	14.60	15.19	15.77	16.35	16.94	17.52	18.11	18.69	19.86	20.44	1168.0
150	8.76	9.39	10.01	10.64	11.26	11.89	12.52	13.14	13.77	14.39	15.02	15.64	16.27	16.90	17.52	18.15	18.77	19.40	20.03	21.28	21.90	1251.0
200	11.68	12.52	13.35	14.18	15.02	15.85	16.69	17.52	18.36	19.19	20.03	20.86	21.69	22.53	23.36	24.20	25.03	25.87	26.70	28.37	29.20	1669.0

bination and optimum *pH* value selected. When making such experiments, the effect of slow stirring, at velocities just high enough to keep the floc in suspension, should be carefully observed. (Tables 226 and 227.)

Quantities of *ferrous sulfate* and *lime* vary greatly. Addition of lime alone, particularly to waters high in magnesia, may coagulate the water more or less perfectly, thereby reducing the dose of ferrous sulfate required. Table 228, based on experience at Cincinnati, and New Orleans (1913–1914), gives the average quantities of ferrous sulfate and lime required. For quantities of chemicals required to soften water, see p. 683.

Table 230. Solubilities of Certain Calcium and Magnesium Salts* in Water at Various Temperatures

t°C.	$CaCO_3$ †[21]	$Ca(OH)_2$ [13]‡	$CaSO_4$ [22]
	Grams per 100 c.c. of water	Grams per 100 grams of water	Grams per 100 c.c. of water
0	0.081	0.185	0.1759
10	0.070	0.176	0.1928
20	0.065	0.165	0.2034§
30	0.052	0.153	0.2090
40	0.044	0.141	0.2097
50	0.038	0.128	0.2038§
60	....	0.116	0.1972
70	....	0.106	0.1891§
80	....	0.094	0.1802§
90	....	0.085	0.1710§
100	....	0.070	0.1619

* Value interpolated from Dupre and Bialas[23] for solubility of $Mg(OH)_2$: 0.009 gram per 100 c.c. of water. (Conductivity method.)

† The solubility of $CaCO_3$ varies greatly with the partial pressure of carbon dioxide in the air with which it is in contact. Wells experimented with water containing 3.02 to 3.27 p.p.m. CO_2.

‡ Average curve from several published results.

§ Interpolated results.

Efficiency of coagulation, is dependent upon (*a*) the thorough dispersion of the applied chemical through the water and (*b*) the optimal exposure of the adsorptive surfaces of precipitated chemicals to the bacteria and other matters, suspended and dissolved, which form flocs with the applied chemicals.

This dispersion of the chemical necessitates its thorough admixture with the water. The process of flocculation involves time for bringing the surfaces of the flocs formed into contact with the matter to be removed. Ultimately an equilibrium is established between the various factors. The time factor usually afforded by a period of detention in a coagulating basin may be reduced by producing contact artificially by slow mixing.

The ideal is to produce dispersion by violent initial mixing and then circulate the water slowly through the coagulating basin (velocity 0.15 to 0.60 ft. per sec.) arranging to leave the requisite amount of floc in the water applied to filters. Practically *mixing* may be accomplished by channels provided with baffles, orifices or a combination of both; mixing tanks provided with mechanical agitators; by passing through pumps, and by the hydraulic jump. Baffled channels require from 1.0 to 3.0 feet head dependent upon construction. No head is lost by passing through pump. For losses in Venturi throats see p.

461. Ellms* at Milwaukee[14] and Hoover† at Columbus[15] showed that the hydraulic jump may be successfully employed with losses in head as low as 0.75 ft.

Mixing tanks for flocculation are usually equipped with slowly-moving paddles for stirring the water as it passes through the tank. A mixing tank is more flexible and is more easily cleaned than a baffled basin. Examples are at South Pittsburgh, Newark, O.,[18] Springfield, Ill., Columbus, O.,[19] and the Glenlyon Print Works, R. I. "Vortex" tanks, in which the inlet pipes enter circular tanks tangentially, thereby imparting a circular or spiral motion to the water, are in use at Providence,[20] R. I., West Palm Beach. Fla., and Oshkosh, Wis.

The **Dorr Agitator,**‡ a form of mixing tank, used at Springdale, Pa., and elsewhere, consists of a suitable tank containing a slowly revolving mechanism, which allows the coagulated particles to settle to the bottom where they are scraped to the center and, by means of an air lift, elevated through a hollow, vertical, central shaft, and distributed evenly over the surface by revolving "launders" whereby a continuous, evenly distributed circulation is kept up, and motion is maintained within the mass, and not only near the bottom and sides, as is the case with tanks provided with insufficient rotating paddles, or paddles operated at too low speed. Another form of this device uses a slowly revolving propeller in a central well instead of the air-lift.

Period of Coagulation. With some waters violent mixing following by a short coagulation period suffices, but in most cases coagulation requires more time, from 1 to 6 hours. The usual practice is to mix for a definite period and then to pass through coagulating basins holding, for treating with sulfate of alumina, the flow for 0.25 to 6 hours, and, for lime-soda treatment, the flow for 4 to 12 hours. Much better results can be obtained with most waters by stirring the treated water 15 to 30 minutes at a velocity just sufficient to keep the floc in suspension, but not great enough to cause them to disintegrate (under which conditions the flocs increase in size),[16] and following this treatment by a period of coagulation of 2 or 3 hrs., rather than depending on a longer period of coagulation alone. Certain waters deflocculate when stored too long after slow mixing. Factors which determine the period of coagulation are the *pH* value of the treated water, the character of the suspended matter, the kinds and amounts of the salts in solution, the temperature and the quantity of coagulant applied. The optimal velocity in the coagulation basin varies from 0.1 to 0.2 ft. per sec. for soft, colored waters, to 0.4 to 0.7 ft. per sec. for hard ground waters, to as high as 1.5 ft. per sec. for turbid, hard surface waters. Low velocities must be maintained between coagulating basins and filters.

Slow vs Rapid Filtration. The period of coagulation for waters purified with modified slow filters, *e.g.*, Washington, D. C., should be long enough to effect the practically complete removal of all coagulant in the basin; usually 24 hr. is more than ample; with mechanical mixing or stirring a shorter period will suffice. Rapid filtration depends for its effectiveness upon proper coagulation. Waters for rapid filters should not be given so long a period of coagu-

* For Cleveland studies, see *Water Works*, July, 1926, 323.
† For use at Bay City, Mich., see *E. N. R.*, Apr. 29, 1926, p. 682.
‡ Made by Dorr Co., New York City.

lation that all floc will be precipitated in the coagulating basin, and none left to form a suitable film in the filter.

Relation between Treatment and Corrosion. Most filtered surface waters are saturated with oxygen and when coagulated at the optimum hydrogen-ion concentration (*pH* 5.0 to *pH* 6.0) they will corrode water pipes and other metal surfaces, causing what is known as "red water." To prevent this, the *pH* value of the water may be raised prior to distribution. This is commonly done by the addition of lime or soda[4] as it enters the filtered water reservoir.*

Recently *sodium silicate*† has been used in England and in the United States (Milford, Mass.), to prevent corrosion. This chemical is decomposed by free carbon dioxide depositing a silicious protective coating on metal surfaces. At Milford-Hopdale, Mass., the addition of 3 to 4 p.p.m. commercial silicate containing about 38 per cent. silicon dioxide (SiO_2) inhibited the solvent action of the water of lead services more effectively than the addition of 11 p.p.m. quicklime, and without increasing the hardness of the water or robbing it of its agreeable carbonic acid. See Weston, J. A. W. W. A. Vol. 15, 1926.

DISINFECTION

Disinfection is practiced to destroy disease germs. Total destruction of all bacteria by *sterilization* is not usually necessary. Proper filtration should give a water free from harmful bacteria, but as an additional precaution it is customary to treat the filtrate with a disinfectant. While the disinfection of the water is highly desirable, it is only an added safeguard and not a substitute for proper and efficient filtration. Disinfection may be accomplished by heat, light, application of chemicals including ozone, and by ultra-violet rays.

Heat is an efficient disinfecting agent, but too costly where large volumes have to be treated; boiled water has a flat, insipid taste, caused by the loss of dissolved gases. To sterilize water completely, it should be boiled ½ hr.; disinfection may be accomplished by heating to 60°C. for 15 min. or by boiling for a shorter time. Sterilization may be accomplished also in sterilizers designed on the countercurrent principle. In the Forbes sterilizer, the water is slowly heated, boiled a few seconds, and then cooled again. The hot water leaving the sterilizer is cooled by the raw water entering. The flow of water is induced by the expulsion of part of the water during the act of boiling; consequently there is no flow through the apparatus when the heat is shut off. Water must be heated to the boiling point to pass through the apparatus. The original dissolved gases and taste of the water are retained and the temperature raised less than 7°C. (13°F.).

Light. Sunlight exerts a powerful germicidal action on bacteria in the surface waters of reservoirs. This action, even in clear waters, is of no importance at depths of more than a very few feet, unless the water be in active circulation and the action of light be exerted on successive surface layers. Light has no practical effect in turbid or highly colored waters.

* The optimum *pH* value for coagulation is frequently less than 6.0, while the *pH* value to inhibit corrosion may be as high as 8.0. Consequently alkali may not be added to water before filtration in such cases.

† Known as water glass.

Chlorine* is the most widely used disinfectant. It is supplied in the liquid form or combined with lime or soda as bleaching powder or sodium hypochlorite. It has both germicidal and oxidizing properties.

Bleaching powder,† sometimes called chloride of lime, chlorinated lime, or calcium hypochlorite, is made by saturating quicklime with chlorine. It is an active sterilizing and oxidizing agent and usually contains 30 per cent. of impurities, chiefly lime and water. It loses strength rapidly if exposed to the air. Its available ingredients have the formula $CaCl_2 + Ca(OCl)_2$. The $CaCl_2$ is inactive, but the $Ca(OCl)_2$ is decomposed by the carbonic acid in the water into hypochlorous acid (HOCl) and $CaCO_3$. The former compound is unstable and gives up its oxygen to organic matter with the production of hydrochloric acid, which latter is neutralized by the alkalinity of the water. Strength of bleaching powder is measured in terms of available chlorine. The actual oxygen liberated according to the formula is 22.5 per cent. of the available chlorine. Bleaching powder should contain not less than 35 per cent. of available chlorine.

Sodium hypochlorite, prepared by electrolyzing a solution of sodium chloride, has been used in place of a solution of bleaching powder. The brine to be electrolyzed is led through a special electric cell where a complicated reaction takes place, which may be partly represented by the following formula:

$$NaCl + H_2O + \text{current} = NaOCl + H_2$$

The solution discharged from the cell may be applied to water like any other chemical solution. One hp. per 24 hr. can produce from 10 to 12 lb. of chlorine, as NaOCl, requiring the use of 25 to 30 lb. of salt.

Chloramine disinfection‡ has received considerable attention. The process consists in combining bleaching powder with ammonia just before applying it to the water to be treated, resulting in the following reaction:

$$\underbrace{2CaOCl_2}_{\text{Bleach}} + \underbrace{2NH_4OH}_{\text{Ammonia}} = \underbrace{2NH_2Cl}_{\text{Chloramine}} + 2H_2O + CaCl_2 + CaO_2H_2$$

While efficient, and while possessing some advantages over chlorine, chloramine is more difficult to apply and more expensive.

Chlorine gas is the most convenient and most used disinfectant. It is a greenish-yellow gas, easily compressed to a liquid. Under a pressure of 6 atmospheres, it liquefies at 0°C. The use of chlorine gas as a sterilizing agent was introduced by Maj. C. R. Darnall in 1910. It is marketed in cylinders containing practically pure chemicals. These cylinders hold from 50 to 250 lb. of chlorine. Larger cylinders and tank cars have been suggested for large users. Pressure in the cylinders varies between 50 and 100 lb. per sq. in. The application of chlorine gas is much more facile than the application of bleaching powder; the apparatus requires much less room, and the discomfort and corrosion due to dust and fumes are avoided. Chlorine gas may be produced by the electrolysis of brine in special cells and frequently at a considerable saving in cost. There are installations at Montreal, Little

* For additional information see Race, "Chlorination of Water" (Wiley, 1918); also E. B. Phelps, "Chlorination of Water and Sewage," J. Boston Soc. C. E., Vol. 23, No. 2, 1926.

† The other halogens, bromine, fluorine, and iodine, have analogous properties.

‡ See Hale, "The Chloramine Process as Applied to the Catskill Water." *J. A. W. W. A.*. Vol. 6, 1919, p. 804.

Falls, and elsewhere. Theoretically, chlorine gas is three times as strong as bleaching powder; in practice, it is difficult to use all the chlorine in bleaching powder; 1 part of chlorine gas may do the work of 4 or 5 parts of bleaching powder. Chlorine gas costs from four to six times as much, but this cost is likely to decrease with the demand and economies in manufacture. The cost of disinfecting water with chlorine gas varies between 10 cts. and 50 cts. per mg. Ordinarily from 0.2 to 0.5 p.p.m. of chlorine is sufficient to disinfect a water, although some waters require more. Gas masks should be provided where chlorine is applied under pressure.

Chlorination Tastes and Odors.* Chlorination has at times been the apparent cause of objectionable tastes and odors in the water. These tastes and odors are of three kinds: (1) tastes and odors of the chlorine itself produced by overdosing; (2) tastes and odors caused by the killing of microscopic organisms, liberating the essential oils, and by the production of organic compounds by the reaction between chlorine and soluble organic matter present in the water; (3) combinations of chlorine and organic compounds, *e.g.*, phenol, acetone, and vegetable matter. Lederer and Bachmann[5] state that 0.6 p.p.m. is smallest quantity of Cl which can be detected by the ordinary observer. The proper amount of chlorine can be controlled by maintaining from 0.1 to 0.3 (usually 0.2) p.p.m. excess after 5 min. absorption.†

By using an excessive dose of chlorine on the Catskill supply in New York City, tastes and odors due to the killing of Synura were destroyed.[6]

Objectionable taste due to the compounds which chlorine forms with organic matter may be avoided: (1) by the proper proportioning of the dose; (2) by storage and aeration after treatment, which gives an opportunity for the decomposition of the objectionable chlorine compounds; (3) by filtration; (4) by the addition of sodium thiosulfate, after a sufficient time has elapsed to enable the chlorine to act. Thiosulfate neutralizes chlorine action.

Table 231. Weights of Chlorine‡

Datum	Gaseous chlorine§	Liquid chlorine
Specific gravity (At. Wt. = 35.46)	3.221 (Air = 1)	1.44 (Water = 1)
Weight of 1 liter	3.221 grm.	1440 grm.
Weight of 1 cu. ft.	0.201 lb.	89.752 lbs.
Weight of 1 gal.	0.0269 lb.	11.999 lbs.

One volume of liquid chlorine is equivalent to 444.4 volumes of chlorine gas.

Table 232. Solubility of Chlorine[7]

Temperature		Solubility ratio by‖ volume	Pounds of chlorine soluble in 1,000,000 gals. of water
C.°	F.°		
10	50	3.095	83,196
20	68	2.260	60,838
30	88	1.769	47,753

* See "Problems in the Chlorination of Water," *E. N. R.*, Vol. 87, 1921, pp. 392, 444.

† See Papers by Wolman, *Jl. Ind. Eng. Chem.*, Vol. 11, March, 1919, and *E. N. R.*, Apr. [illegible]921, p. 639; and Hale, *J. A. W. W. A.*, Vol. 10, 1923, p. 247.

‡ Smithsonian Tables, 1921.

§ Atmospheric pressure, sea level 32°F.

‖ See also Seidell's "Dictionary of Solubilities," 1919.

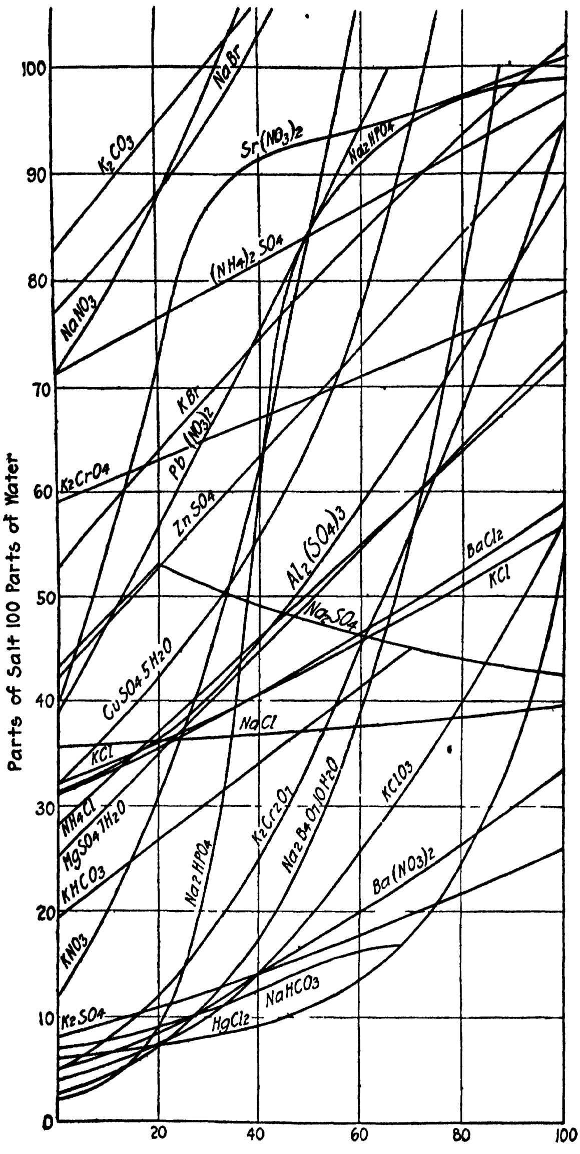


Fig. 258—Solubilities of various salts in water at various t°C

For names of chemical compounds see p 676

Ozone is applied in the form of ozonized air, made by the silent discharge of electricity in a space through which a current of air passes. The air must be dried by refrigeration or other means before ozonization, and the ozonized air must be mixed with the water intimately, by absorbing towers, atomizing sprays, or multiple diffusers. The process is ideal in that sterilization is effected without the addition of any foreign matter. The largest installations are in Petrograd and at St. Maur waterworks, Paris. Devices for producing ozone are expensive and the cost of treatment is high. The process is inefficient when the water is turbid or colored, and the treatment of a well-purified filter effluent is seldom necessary. As a disinfectant, ozone is far inferior in efficiency and economy to chlorine.

Ultra-violet Rays. Ultra-violet rays from a mercury vapor quartz lamp will sterilize water completely and rapidly—provided it be exposed to the rays in thin layers—contains no interfering turbidity or color, and is not supersaturated with gases. Ultra-violet rays not only destroy bacteria in the vegetative stage, but spores as well. The lamps employed are so constructed that nearly all the light produced will enter the water. Usually a number of lamps are inserted in a properly baffled casing so as to give the water a number of exposures between inlet and outlet. Water may be overdosed with ultraviolet rays with impunity. The method has been used in Marseilles, France, since 1910, and has been employed in the United States especially for sterilizing spring and bottled water. Ultra-violet rays cannot compete with chlorine in cost. According to von Recklinghausen,[8] water in large plants would require a current consumption of from 50 to 125 kw.-hr. per mg. Ultra-violet rays and ozone are ideal for the treatment of small volumes of clear water, where the cost of operation is not an important consideration. Details of apparatus may be obtained from the R. U. V. Co., New York City.

Iodine.[17] Vergnoux recommends a 10 per cent. solution, with 0.5 per cent. KI in 80 per cent. alcohol. Five drops of this solution is sufficient for 1 liter of water. A few drops of saturated sodium thiosulfate solution destroy the excess iodine. Penan added a special tablet, containing 0.85 g. of iodine, to 200 liters of water. After 30 min. action, 2 g. of sodium thiosulfate are added to neutralize the residual iodine.

ALGICIDES

Copper Sulfate. The commercial salt has the formula $CuSO_4 + 5H_2O$, and is known as "blue vitriol." Ferrous sulfate containing 1 per cent. of copper sulfate has been used at Marietta, Ohio, as a coagulant. Ten p.p.m. of copper sulfate will destroy *Bact. coli.*

Doses of *copper sulfate* required for different organisms, according to Kellermann[9] are given in Table 241A. Doses should be increased or decreased 2.5 per cent. for each °C. below and above 15°. If organic matter and alkalinity be high, or if carbonic acid be low, the doses should be increased further. Kellermann recommends the limits given in Table 241A. Combinations of copper sulfate and chlorine have been used effectively for the disinfection of swimming pools, as well as for killing microscopic organisms. Dr. F. E. Hale reports death of trout on application (to Esopus Creek, N. Y.) of 0.4 p.p.m. liquid chlorine, thoroughly mixed. Use of 1 p.p.m. available chlorine in

Croton system drove away trout. Weigelt states that 0.5 p.p.m. chloride of lime (0.17 p.p.m. chlorine) kills trout in 3 hr. Goldfish are very susceptible to change of water constituents. Schwartz and Nachtigall, Hamburg, report eels, goldfish and perch unaffected up to 1.6 p.p.m. available chlorine in 6 days (18°C.), 2.5 p.p.m. killed them on second day. 8 p.p.m. HCl kills sunfish and bull minnows in 40 hr.; 12 p.p.m. H_2SO_4, in 24 hr.[10]

Table 233. Safe Limits for Treating Water with $CuSO_4$ to Prevent Killing Fish

Name of fish	Parts per million	Pounds per mill. gals. (approx.)	Name of fish	Parts per million	Pounds per mill. gals. (approx.)
Trout	0.14	1.2	Perch	0.75	6.0
Carp and Suckers	0.30	2.5	Sunfish	1.20	10.0
Catfish and Pickerel	0.40	3.5	Black bass	2.10	17.0
Goldfish	0.50	4.0			

Applying copper sulfate is usually effected by placing in a bag, perforated bucket, or wire basket, attached to a rope and dragged back and forth at the stern of a boat. One or more bags may be used at the same time. Speed of boat and rate of addition of chemical to the bucket or bag may be so regulated that not over 100 lb. will be dissolved in an hour. The boat should move fast enough to avoid too great concentration near the bag, that the killing of fish may be avoided. It is better to apply the chemical when the wind is blowing. Before applying, the body of water should be roughly surveyed in sections, and the volume and dose for each section determined.

APPLICATION OF CHEMICALS*

Principles. It is essential that the chemical or disinfectant whether added in solution, in the dry, or in the gaseous form, be accurately proportioned to the quantity of water treated. The flow of the treated water being known, the dose of chemical may be varied by hand from time to time, or automatically by numerous devices.

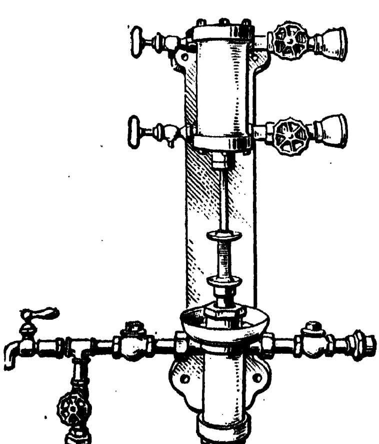


FIG. 259.

The entering chemical should be mixed quickly with the raw water. Diffusion assists this process materially. See *Mixing*, p. 660.

Solutions, Reciprocating Pump. Where a reciprocating pump is used to lift the water, a small coagulant pump may be attached to the valve rod, or a Hodkinson, variable-stroke, synchronous, chemical feed pump may be used. Figure 259 shows this pump, which is a hydraulic pump actuated by the hydraulic pressure generated alternately from the two ends of the reciprocating pump chamber. The stroke of this pump can be varied. Where the water to be coagulated is discharged through a pipe under pressure, the coagulant feed-

* For water softening, see p. 683.

pump may be operated by a turbine placed within the pipe. This device is not reliable for greatly varying flows. Where *chemical solutions* are discharged into pump suctions, a pump suction box furnished by makers of filter equip-

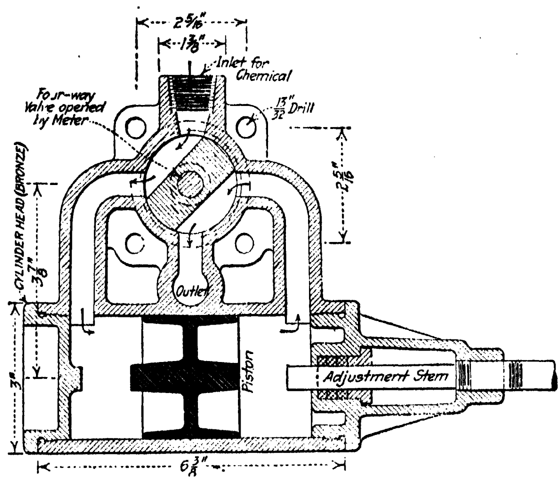


FIG. 260.—Allen chemical feeder. (Oshkosh, Wis.)

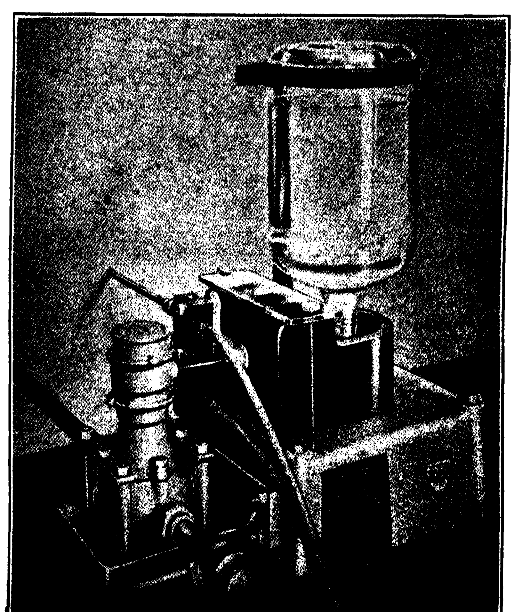


FIG. 261.—Automatic chlorometer for sterilizing small water supplies.
(Wallace & Tiernan Co., Inc.)

ment, can be used to prevent entrance of air. At Oshkosh, Wis., the proportional feeder[11] actuated by the driving stem of a Kreutzberg meter, and shown in Fig. 260, is used. A similar meter-operated device for applying small volumes of solution (*e.g.*, sodium hypochlorite) is shown in Fig. 261.

Orifice Tanks. Where coagulant is added by gravity, the principle of an orifice discharging from a tank in which the solution is maintained at a constant level, may be employed. One or more orifices can be used to discharge the solution into the pipe discharging into the water to be treated. Instead of

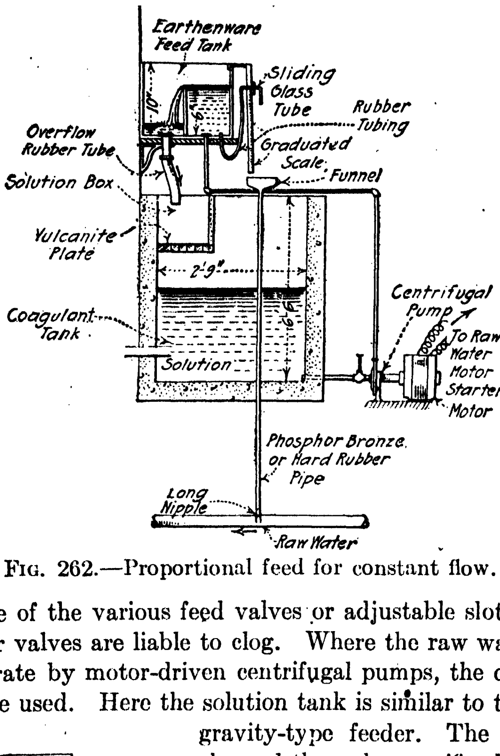


FIG. 262.—Proportional feed for constant flow.

the orifices, one of the various feed valves or adjustable slots may be used. Small orifices or valves are liable to clog. Where the raw water is delivered at a constant rate by motor-driven centrifugal pumps, the device shown in Fig. 262 may be used. Here the solution tank is similar to that used in the gravity-type feeder. The solution is discharged through an orifice box by a motor-driven pump which is started and stopped by the same switch which controls the raw-water pump. The solution overflows from the feed tank whenever the raw-water pump is operated. Attached to the feed tank is an outlet of some sort. The one shown in Fig. 262 consists of a flexible tube connected with a calibrated glass tube which can be raised and lowered on a graduated scale. This tube discharges into a pipe connected with the raw-water main. Whenever the raw-water pump stops, the coagulant pump stops also, the feed tank quickly drains and the flow stops; and *vice versa* when the raw-water pump starts.

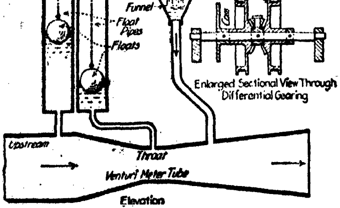


FIG. 263.—Diagrammatic view of Venturi chemical feed device.

Automatic Feeds. The first *automatic chemical feed devices* based on the Venturi meter were installed at Columbus, Ohio, in 1908. In this device the floats actuated by the meter tube operated a cam, which in turn controlled

the orifice discharging from a constant-head tank. Recently, at Baltimore, the cam has been used to start and stop a small electric motor, which in turn controls the raising and lowering of the plug in a circular orifice discharging from a constant-head tank. At Cairo, Egypt, the plug is raised and lowered directly by means of the cam. Figure 263 illustrates this type of apparatus. It is made by the Builders Iron Foundry. Earl devised similar equipment for New Orleans.

At Toronto, Ont., Gore designed an effective device for feeding large volumes of sulfate of alumina solution.[12] This device consists of the following parts: (*a*) solution tanks for preparing a saturated solution of chemical; (*b*) diluting device for preparing solutions of varying strength at will; (*c*) a measuring tank for proportioning the discharge of solution to the flow of water.

The solution tanks provide for an intermittent supply of chemical and a constant supply of water. The diluting device comprises a large, hydrometer-like float, placed in a mixing tank, and controlling by its position, inlet valves for solution and diluting water, at top and bottom, respectively. The float is counterbalanced by a beam with sliding weight so that any desired concentrations of solution may be prepared by setting the weight. Because of the difference in specific gravity between solution and water, good mixing is obtained in the tank. The measuring tank consists of a vessel with an adjustable orifice which is adjusted to the flow of water by means of a double electrical relay passing through a solenoid controlling the inlet and exhaust valves of an hydraulic cylinder, the piston of which connects with the rod for adjusting the size of the orifice.

Effect of Chemicals upon Construction Materials. Strong solutions of sulfate of alumina, hypochlorite of lime, or bleach, rapidly corrode iron, steel, and copper, but brass, bronze, and tin resist well. Most woods are quickly destroyed, if unprotected. Dense Portland cement mortar, slate, pottery, paraffin, hard rubber and lead, are but slightly attacked, if at all.

Tanks holding *acid coagulant solutions* should be built of concrete or enamel-lined iron, and the piping should be of rubber, glass, earthenware, or acid-resisting metal. Phosphor-bronze, "Duriron," "Alcumite" and Monel metal, withstand sulfate of alumina well. Alkaline solutions may be stored

Table 234. Rates of Flow of Chemical Solutions of Various Strengths for Various Rates of Filtration

U. S gallons		Gallons of chemical solution per min to add 10 parts of chemical per million						
24 hours	Min	1%	2%	3%	4%	5%	6%	10%
250,000	174	0 174	0 087	0 058	0 044	0 035	0 029	0 017
500,000	347	0.347	0 174	0 116	0 087	0 069	0 058	0 035
1,000.000	694	0.694	0.347	0 231	0.174	0 139	0 116	0.069
1,500,000	1,042	1.042	0.521	0 347	0 260	0.208	0 174	0.104
2,000,000	1,389	1.389	0.694	0.463	0 347	0 278	0 232	0.139
2,500,000	1,736	1.736	0.868	0 579	0 434	0.347	0.289	0.174
3,000,000	2,083	2.083	1.042	0 694	0 521	0.416	0 347	0.208
4,000,000	2,778	2.778	1.389	0 926	0.694	0.556	0.463	0.278
5,000,000	3,472	3.472	1.736	1 157	0.868	0.694	0.579	0.347
7,500,000	5,208	5.208	2.604	1 736	1.302	1.042	0.868	0.521
10,000,000	6,944	6.944	3.472	2.315	1.736	1.389	1.157	0.694
20,000,000	13,889	13.889	6.944	4.630	3.467	2 778	2.315	1,389

in iron or concrete tanks and fed through iron piping. To prevent clogging, pipes handling lime or milk of lime should be of ample diameter and provided with numerous cleanouts. For pumping milk of lime, centrifugal pumps should have white iron casings and a monel-metal shaft.[17] Tanks for bleach solution are best made of reinforced concrete or pure wrought iron. Piping may be of pure black wrought iron, lead, stone-ware, or glass; fittings should be of stone-ware or acid-proof bronze. Avoid wood, copper, ordinary brass, and steel. Red brass has been used recently.

Dry Chemicals. In most plants, dry feeding of chemicals is practicable. Dry feeding avoids solution tanks, orifices, and pipes, and economizes space.

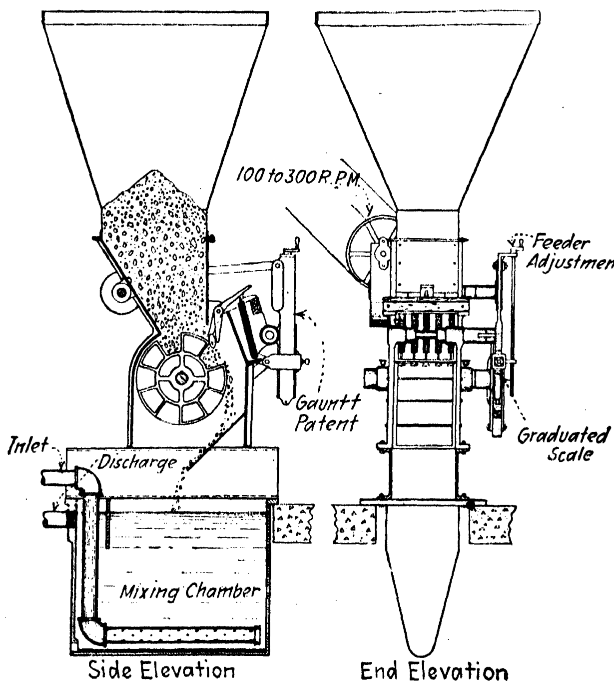


FIG. 264.—Lump alum feeder.
(Gauntt.)

It permits a ready control of applied chemicals. Sulfate of alumina, quicklime, hydrated lime, and ferrous sulfate may be fed dry if the chemicals are ground and means are provided to crush any lumps which may form during storage. Ferrous sulfate, particularly, is apt to cake when stored in bins; it should therefore be freshly crushed and fed quickly.

Commercial Types. One type of dry feeder consists essentially of a hopper from which a small helical conveyor, driven by electric or hydraulic motor, and connected with devices for varying speed, discharges various quantities of coagulant from the hopper into a stream of water leading to the point of application. This type of apparatus is used at Knoxville, Tenn., and Springfield, Mass., and is typified by the Gauntt and American Water Softener Co.'s

feeders. Other feeders are made with revolving cylinders containing depressions which are alternately filled and emptied as the cylinder revolves below the hopper. The Gauntt* feeder for lump alum (Fig. 264) is typical.

The Booth,† Pittsburgh, and other feeders (Fig. 265) make use of a revolving disk upon which a hopper discharges the chemical, and from which a scraper diverts it to a stream of water leading to the point of application.

Most dry feeders are capable of automatic, proportional discharge by the proper connections with the raw-water Venturi.

Fig. 265.—Dry feeders, Oklahoma City.
(American Water Softener Co.)

F. B. Leopold has devised a combined feeder and hydrating apparatus for dry quicklime consisting of feeder, hydrating apparatus, and mixing tank, thus avoiding the measurement of milk of lime (Fig. 267).

The maximum and minimum capacities of the W. and T. dry chemical feeder† of the low-speed type with various chemicals are given in the following table (Table 235). There is another feeder of the high-speed type which has several times the capacity of the small feeder. Feeders of this type are especially adapted to feeding crystalline chemicals. The other dry feeders, like the Gauntt, Pittsburgh, International, etc., are also built in several sizes.

* W. J. Savage, Inc., Knoxville, Tenn.
† Made by Wallace and Tiernan Co.

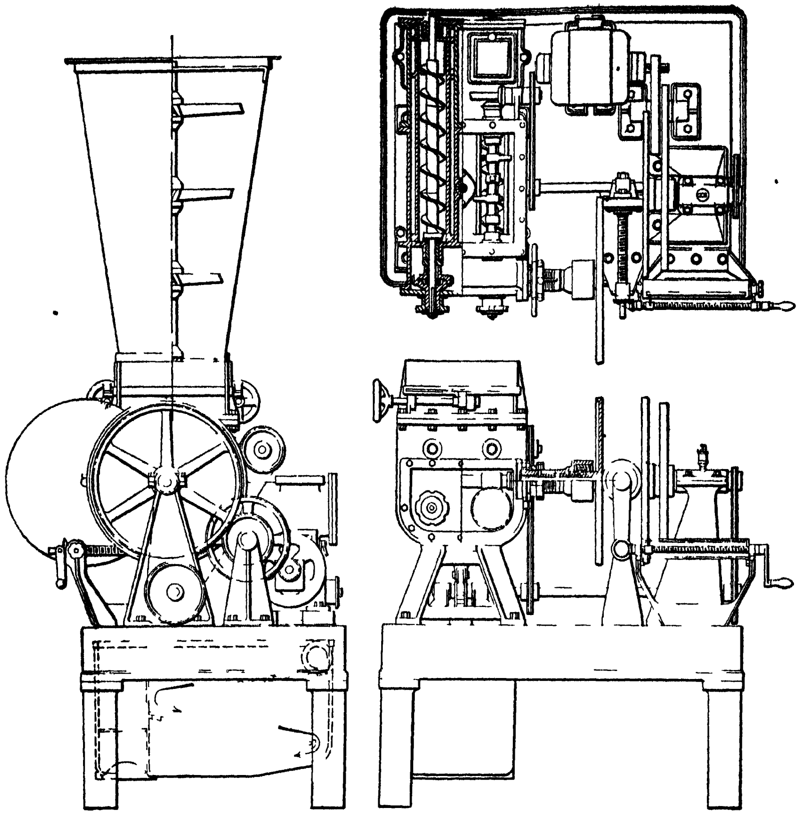


FIG 266—Dry feeder, American Water Softener Co

Table 235. Discharges of Low-speed Feeder (Wallace & Tiernan Co.)

Speed of Driving Motor—1200 R p m Power Required ¼ H p Belt 1 Inch Flat

Pulley size 3" × 1¼" on motor 12" × 1¼" on feeder Feed table speed—¼ r p m					Pulley size 8" × 1¼" on motor, 8" × 1¼" on feeder Feed table speed—1 r p m				
Chemical	Rate of feed lbs per hour, Min	Rate of feed lbs per hour, Max	Water pressure for ejector	Quantity water for ejector	Chemical	Rate of feed lbs per hour, Min	Rate of feed lbs per hour, Max	Water pressure for ejector	Quantity water for ejector
Hydrated lime	0.5	12	Fifteen pounds per square inch	Twelve gallons per minute	Hydrated lime	5	48	Fifteen pounds per square inch	Twelve gallons per minute
Soda ash	1.0	20			Soda ash	6	90		
Potassium alum	1.25	20			Potassium alum	5	90		
Hydrated iron sulfate	1.5	25			Hydrated iron sulfate	10	100		
Flowers of sulfur	2.5	28			Flowers of sulfur	7.5	150		

NOTE—All data for operating water pressure and quantity of water required for the ejector are based on the assumption that there is no back pressure in the discharge line and that there is a fall of at least 1 ft per each 25 ft of discharge line from the ejector to the point of application.

Where large quantities of lime or soda are used for water softening, automatic weighing devices (discharging intermittently), may be used in place of continuous dry feeders.*

Lime should be slaked in hot water. W. F. Monfort has advised an initial temperature of 160°F. for the hot water supply and 200°F. in the slaking tanks. Slaking tanks should be provided with stirrers. From 90 to 95 per cent. of the available, soluble lime may be added to the water. The rest is lost in the sludge remaining in the slaking tanks. It may be added either in the form

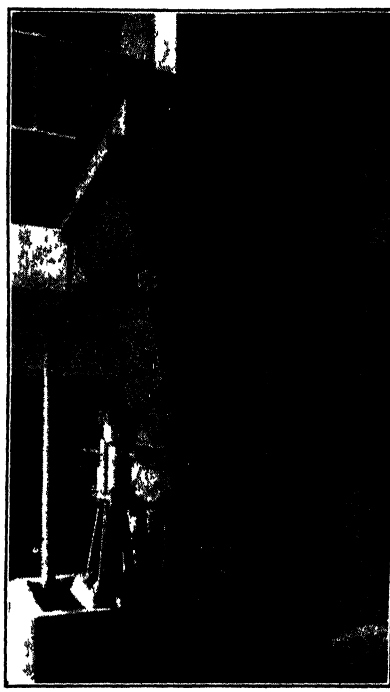


FIG. 267.—Dry-feed machine, type XL.
(Municipal Service Co.)

of milk of lime (at St. Louis 3 25 lb. of water to 1 lb of lime) or in the form of a saturated or partly saturated lime water. It is essential that the strength of lime solutions should be uniform.

Bleaching powder is difficult to apply to water. It is first mixed with a smaller quantity of water—not less than 0.5 gal. water per lb. of bleach—and this "paste" thoroughly mixed and diluted to make a solution containing not over 2 per cent. of bleach. Considerable sludge is produced, and the chlorine in this should be recovered by agitating it with water and using the water after the subsidence of the sludge to make up the next batch of solution. The solution shall be constantly agitated until applied to the water.

Liquid chlorine is added directly from cylinders by means of a regulating apparatus.† Figures 268 and 269 show types of an apparatus combined with

* For Columbus equipment, see 1914 Report, Division of water, City of Columbus.
† Chlorine equipment is made by Wallace and Tiernan Co., Newark and Paradon Co., New

platform scale for convenience in controlling the flow. Pressures in cylinder vary from 110 lb. downwards. The gas enters through an inlet valve which adjusts its opening in inverse ratio to the entering gas pressure. The chlorine then passes to a fixed orifice which both measures and regulates the flow, employing the principle of pressure drop across the orifice, which drop is indicated by a long manometer tube calibrated to read in lb. per 24 hr. After

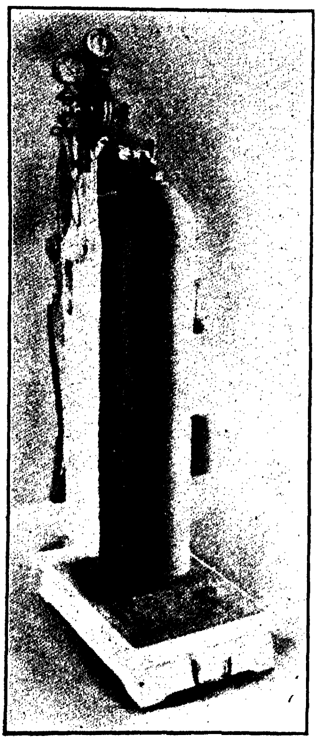


FIG. 268.—Chlorinator combined with platform scales.
(Paradon Co.)

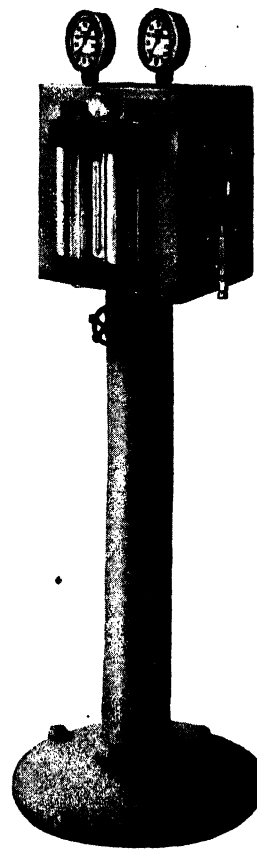


FIG. 269.—Manual control solution feed chlorinator, type MSP.
(Wallace & Tiernan Co., Inc.)

discharge through the orifice, the gas is maintained at constant meter pressure by a diaphragm-controlled valve. The apparatus feeds chlorine either directly or through an injector supplied with water at 20 lb. or more pressure. An automatic chlorine-shut-off valve stops the flow of gas instantaneously with the stoppage of the injector supply. The formation of chlorine hydrate with water of low temperatures* and resulting stoppage and backflooding of feeder are prevented by using the injector type of feeder. Sudden expansion may

* Below 9°C.

cause chlorine to cool with loss in pressure. Artificial heat may be necessary to overcome this effect, and prevent the formation of chlorine hydrate.

Apparatus may be designed to maintain a proportional flow of chlorine and water, making use of the differential between the upstream and throat heads in a Venturi meter to actuate the controlling mechanism, either directly or through hydraulic or electrical relays.

The vacuum chlorinator of Wallace and Tiernan Company (Fig. 270), is recently developed apparatus. The chlorine is always under a vacuum, and its flow automatically ceases when the water supply to the apparatus is shut

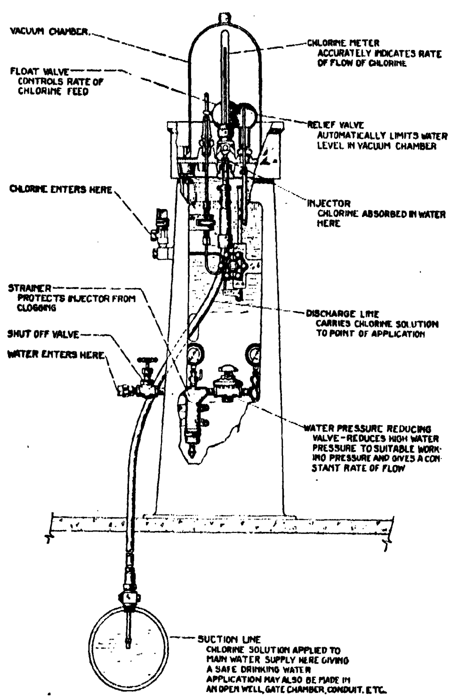


FIG. 270.—Manual control chlorinator solution feed, type MSVM. (Wallace & Tiernan Co., Inc.)

off. By controlling the rate of flow of the auxiliary water supply, the rate of application may be proportioned to the flow of water.

The vacuum produced by the flow of the auxiliary supply of water through the ejector draws the chlorine into the ejector where it is mixed with the auxiliary water supply and is conveyed to the point of application.

The suction created by the water passing through the ejector is transmitted through a silver adjustable head tube to the inside of a bell jar which stands in a partially filled tray of water. The vacuum created causes the level of the water inside the bell jar to rise above the water level outside the jar. This opens the float reducing valve which allows the chlorine to enter the bell jar, the chlorine cylinder being connected directly to the feed line to which this float valve is attached.

A glass meter tube with a calibrated orifice at the top sits over the silver adjustable head tube. The only way that the chlorine can enter the ejector is through the orifice at the top of the meter tube and down through the silver

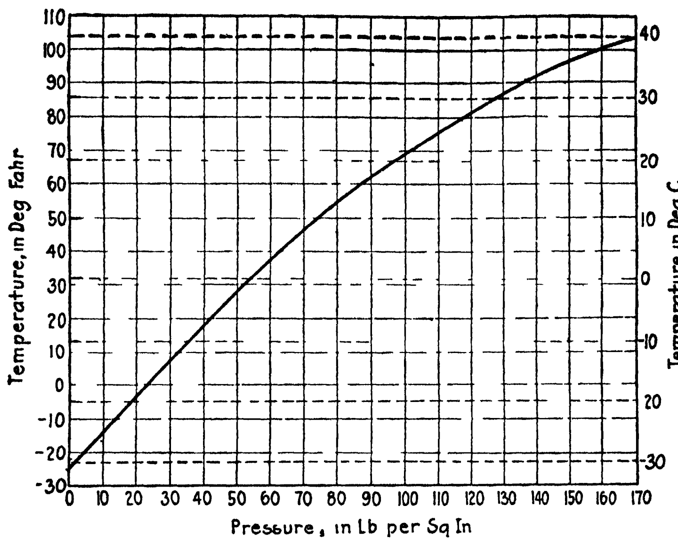


FIG. 271 —Curve showing pressure of chlorine gas at various temperatures

tube. The flow of gas depends on the drop in pressure across the orifice The flow is determined by measuring this drop which is equal to the difference in height of the water inside the bell jar and inside the meter tube

Table 236. Chemicals and Formulas Commonly Used in Water Purification Work

Substance	Formula	Substance	Formula
1. Acids		**2 Salts — (Continued)**	
Acetic	CH_3COOH	Magnesium carbonate	$MgCO_3$
Hydrochloric	HCl	chloride	$MgCl_2$
Nitric	HNO_3	sulfate	$MgSO_4$
Oxalic	$H_2C_2O_4$	Manganous sulfate	$MnSO_4$
Sulfuric	H_2SO_4	Mercuric chloride	$HgCl_2$
2. Salts		Phenol	C_6H_5OH
Alcohol	CH_5OH	Potassium bichromate	$K_2Cr_2O_7$
Aluminum hydrate	$Al(OH)_3$	carbonate	K_2CO_3
sulfate	$Al_2(SO_4)_3$	chloride	KCl
Ammonium chloride	NH_4Cl	chromate	K_2CrO_4
ferrous sulfate	$(NH_4)_2Fe(SO_4)_2\ 6H_2O$	hydrate	KOH
oxalate	$(NH_4)_2C_2O_4\ 2H_2O$	hydrogen sulfate	$KHSO_4$
persulfate	$(NH_4)_2S_2O_8$	iodide	KI
Barium carbonate	$BaCO_3$	nitrate	KNO_3
chloride	$BaCl_2$	nitrite	KNO_2
sulfate	$BaSO_4$	oxalate	$K_2C_2O_4\ 2H_2O$
Calcium carbonate (calcite)	$CaCO_3$	permanganate	K_2MnO_4
chloride	$CaCl_2$	platinic chloride	K_2PtCl_6
oxide	CaO	sulfate	K_2SO_4
sulfate	$CaSO_4\ 2H_2O$	sulphocyanide	$KSCN$
Chloroform	$CHCl_3$	Silica	SiO_2
Cobaltous chloride	$CoCl_2\ 6H_2O$	Silver chromate	Ag_2CrO_4
Copper sulfate	$CuSO_4$	nitrate	$AgNO_3$
Ferric chloride	$FeCl_3$	sulfate	Ag_2SO_4
oxide	Fe_2O_3	Sodium bismuthate	$NaBiO_3$
sulfate	$Fe_2(SO_4)_3$	Sodium carbonate	Na_2CO_3
Ferrous carbonate	$FeCO_3$	chloride	$NaCl$
oxide	FeO	nitrate	$NaNO_3$
sulfate	$FeSO_4$	nitrite	$NaNO_2$
Hydrogen peroxide	H_2O_2	sulfate	Na_2SO_4
Lead acetate	$Pb(C_2H_3O_2)_2$	thiosulfate	$Na_2S_2O_3$

Table 237. Principal Chemicals Used for Water Purification

Trade name	Strength	Formula of active ingredients	Price per 100 lb. delivered†
Basic sulfate of alumina*	16–22% Al_2O_3	$Al_2(SO_4)_3$, XH_2O‡ + XAl_2O_3	$1.10–$1.60
Ferrous sulfate	95–100% $FeSO_4$, $7H_2O$	$FeSO_4$, $7H_2O$	0.65– 0.75
Copper sulfate	99% $CuSO_4 + 5H_2O$	$CuSO_4$	4 50– 5.50
Quicklime...	75–99% CaO	CaO	1 00– 1.50
Hydrated lime	80–99% $Ca(OH)_2$	$Ca(OH)_2$	0 90- 1.50
Soda ash . .	58% Na_2O	Na_2CO_3	1 00- 1 85
Caustic soda	70–76% Na_2O	NaOH	3 00- 4.00
Barium carbonate	98% $BaCO_3$	$BaCO_3$	3 00– 4 00
Bleaching powder	30–38% Cl	$CaCl_2$, $Ca(OCl_2)$	1 50– 2 50
Liquid chlorine	99 9+ % Cl	Cl_2	4 00–8 00

* Called "alum" or "filter alum"
† Normal ranges of prices
‡ X means uncertain number of molecules, the substance is a mixture

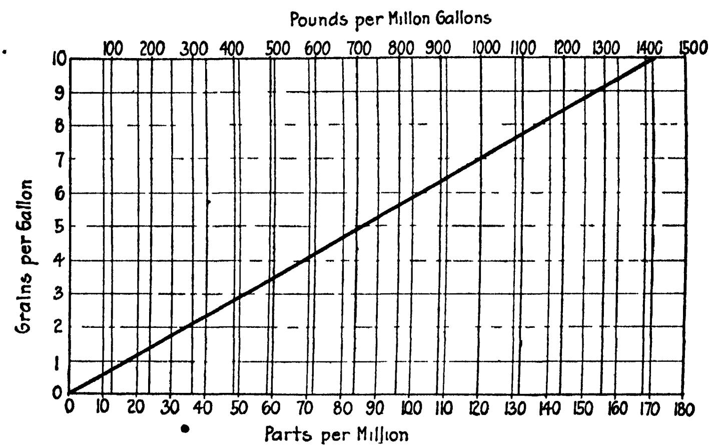


FIG. 272.—Conversion diagram. Parts per million, pounds per million gallons, grains per gallon.

Table 238. Size of Packages for Water Purification Chemicals

Chemical	Kind of package	Weight of chemical in package		Dimensions of package
		Lbs	Kg	
Bleaching powder	Cylinders or 'Drums'	300	136 0	21" diam × 34" high
Bleaching powder	Cylinders or "Drums"	100	45 5	13½" diam × 28" high
Chlorine	Steel bottle†	150*	68 8	9" diam × 4'2" high
Copper sulfate	Barrels	450	205 0	24" diam × 30" high
Ferrous sulfate	Barrels	420	191 0	23" diam × 30" high
Lime—hydrated	Bags	135	61 5	42" long × 24" wide
Soda ash	Bags	300	136 0	38" long × 30" wide
Caustic soda—ground	Drums	550	250 0	21" diam × 34" high
Caustic soda—solid	Drums	750	340 0	21" diam. × 34" high
Sulfate of alumina .	Bags	200	91 0	37" long × 26" wide
Sulfate of alumina	Barrels (42 gals)	325 to 400	148 to 182	24" diam × 30" high
Sulfuric acid	Carboys (12 7 gals)	195	88 7	16" × 16"

Normal minimum carload is 40,000 lb net During emergencies, cars are loaded to capacity.
* 275 lb gross.
† Also in tank cars (England).

Table 239. Volume-weights of Chemicals Used for Water Purification*

Chemical	Loose weights				Average compacted weights	
	Min., lb. cu. ft.	Max., lb. cu. ft.	Average, lb. cu. ft.	Average, kg. cu. m.†	Lb. cu.ft.	Kg.cu.m.
Bauxite, run of mine			81.0	1300		
Bauxite, pulverized, 80 mesh			61.0	980		
Bleaching Powder	48	49.0	48.5	777		
"Boothal"			58.0	930		
Chalk—powdered			64.0	1030		
Chlorine—liquid	....	....	89.8	1440	....	....
Copper sulfate			87.0	1390		
Ferrous sulfate	63	66.0	64.0	1030	70.7	1130
Lime—lump	62	66.0	65.0	1040	77.5	1240
Lime—crushed and powdered		....	64.0	1030	73.5	1180
Lime—hydrated	36	45.0	42.0	670	47.5	760
Sodium Carbonate, soda Ash	53	60.0	62.0	995		
Dense Soda Ash	80	90.0	85.0	1362		
Sodium hydrate (caustic soda) in drums, fused	130	133.0	132.0	2118		
Sodium Thiosulfate			87.0	1390		
Sulfate of Alumina, lumps thru 1.5" ring	62	67.0	65.0	1040	72.0	1150
Sulfate of Alumina, crushed	60	63.0	61.0	980	72.7	1160
Sulfate of Alumina (Hoover Alum), porous		47.5	42.0	670		
Sulfate of Alumina (Hoover Alum), dense and brittle	53	66.0	60.5	970		
Sulfuric acid, conc	....	....	114.9	1840	....	....

* Determinations by C. P. Hoover; J. W. Ellms; General Chemical Co.; Pennsylvania Salt Mfg. Co.; Springfield Water Department, and Authors.
† Equals 1000 times apparent specific gravity.

Table 240. Conversion: Grains per Gallon to Parts per Million

	Grains per U. S. gal.	Grains per Imp. gal.	Parts per 100,000	Parts per 1,000,000
1 gr. per U. S. gal	1.000	1.20	1.71	17.1
1 gr. per Imp. gal	0.835	1.00	1.43	14.3
1 part per 100,000	0.585	0.70	1.00	10.0
1 part per 1,000,000	0.058	0.07	0.10	1.0

Table 241. International Atomic Weights (1923) of Elements Occurring in Water Analyses

	Symbol	Atomic weight		Symbol	Atomic weight
Aluminum	Al	27.0	Manganese	Mn	54.93
Arsenic	As	74.96	Mercury	Hg	200.6
Barium	Ba	137.37	Molybdenum	Mo	96.0
Boron	B	10.9	Nickel	Ni	58.68
Bromine	Br	79.92	Nitrogen	N	14.008
Calcium	Ca	40.07	Oxygen	O	16.00
Carbon	C	12.005	Phosphorus	P	31.04
Chlorine	Cl	35.46	Platinum	Pt	195.2
Chromium	Cr	52.0	Potassium	K	39.10
Cobalt	Co	58.97	Radium	Ra	226.0
Copper	Cu	63.57	Silicon	Si	28.1
Fluorine	F	19.0	Silver	Ag	107.88
Hydrogen	H	1.008	Sodium	Na	23.00
Iodine	I	126.92	Strontium	Sr	87.63
Iron	Fe	55.84	Sulfur	S	32.06
Lead	Pb	207.20	Tin	Sn	118.7
Lithium	Li	6.94	Zinc	Zn	65.37
Magnesium	Mg	24.32	....	....	....

Table 241A. Quantities of Copper Sulfate Required for Different Organisms

Organisms	Parts per mil.	Lbs. per mil. gal. of water	Organisms	Parts per mil.	Lbs. per mil. gal. of water
Diatomaceæ:			Cyanophyceæ:		
Asterionella	0.10	0.8	Anabæna	0.10	0.8
Fragilaria	0.25	2.1	Clathrocystis	0.10	0.8
Melosira	0.30	2.5	Cœlosphærium	0.30	2.5
Synedra	1.00	8.3	Oscillaria	0.20	1.7
Navicula	0.07	0.6	Microcystis	0.20	1.7
Chlorophyceæ:			Aphanizomenon	0.15	1.2
Cladophora	1.00	8.3	Protozoa:		
Conferva	1.00	8.3	Euglena	0.50	4.2
Hydrodictyon	0.10	0.8	Uroglena	0.05	0.4
Scenedesmus	0.30	2.5	Peridinium	2.00	16.6
Spirogyra	0.20	1.7	Glenodinium	0.50	4.2
Ulothrix	0.20	1.7	Chlamydomonas	0.50	4.2
Volvox	0.25	2.1	Cryptomonas	0.50	4.2
Zygnema	0.70	5.8	Mallomonas	0.50	4.2
Microspora	0.40	3.3	Dinobryon	0.30	2.5
Draparnaldia	0.30	2.5	Synura	0.10	0.8
Raphidium	0.30	2.5	Schizomycetes:		
Cœlastrum	0.30	2.5	Beggiatoa	5.00	41.5
			Cladothrix	0.20	1.7
			Crenothrix	0.30	2.5
			Leptomitus	0.40	3.3

Bibliography, Chapter 30. Coagulation

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CHAPTER 31

WATER SOFTENING

Hard waters, so named because of their action on the skin, are those which precipitate soap. They form scale or deposits when used in steam boilers. On account of the formation of insoluble compounds with dyes and soaps, they are unsuitable for washing, bleaching, certain kinds of dyeing, soap making, steam making, tanning, wool scouring, and certain kinds of paper making.

Hardness is due to the presence of the bicarbonates, carbonates, sulfates, chlorides, and nitrates of calcium and magnesium, also to acid constituents. When a hard water is boiled, it loses its bicarbonate, or *temporary,* hardness, leaving the "mineral acid," or *permanent,* hardness as well as some hardness due to carbonates which cannot be so removed. The terms *carbonate* and *non-carbonate* hardness are used to distinguish the hardness due to carbonates from that due to sulfates, chlorides, and other salts. *Total hardness* is the sum of temporary and permanent hardness. Waters containing free CO_2 have an apparent temporary hardness. Hardness of water polluted with mine waters* is due partly to free mineral acids.

Kinds of Hardness

- Alkaline Constituents (Temporary hardness)[1]
  - Ca, Mg and Fe Bicarbonates
  - Ca and Mg Carbonates and Hydrates
- Incrustants (Permanent hardness)
  - Ca and Mg
    - Sulfates
    - Chlorides, Nitrates, Silicates
- Acid Constituents (Corrosive: Permanent hardness and scale forming)[1]
  - Sulfuric acid, Al and Fe Sulfates, CO_2

(Carbonate hardness) (Non-carbonate hardness)

Hardness may be *expressed* in various ways, preferably as parts of $CaCO_3$ per million, Table 242.† Clark, the discoverer of the softening process, in 1841, established degrees of hardness as grains of $CaCO_3$ per Imp. gal.[2] German degrees designate parts of CaO in 100,000 and French degrees parts of $CaCO_3$ in 100,000 of water. What would be called hard water in New England (*e.g.,* Reading: 120 p.p.m.), would be called soft in the limestone regions of the West.

* For barium treatment to remove sulfuric acid, see *E. N. R.*, Apr. 19, 1923, p. 698

† Recommended by Committee, A. P. H. A.

Table 242. Equivalents of Methods of Expressing Hardness

Standard	Clark degrees	German degrees	French degrees	Parts per million	Grains per U.S. gal.
Clark degrees	1.0	0.80	1.43	14.3	0.83
German degrees	1.24	1.0	1.78	17.9	1.04
French degrees	0.70	0.56	1.0	10.0	0.583
Parts per million (milligrams per liter)	0.07	0.056	0.10	1.0	0.058
Grains per U.S. gal	1.20	0.96	1.71	17.1	1.0

Effects. General objections to hard waters are difficulty and disadvantage of their use with soap; difficulty, cost, and danger attendant upon their use for steam making.

Use with Soap. According to Whipple,[3] 1 lb. of average soap will soften 167 gal. of water when hardness is 20 p.p.m., but only 40 gal. when hardness is 100 p.p.m.

Cost of soap per 1,000,000 gal. = $10† × hardness (p.p.m.).

This cost of soap is a measure of the depreciation due to hardness. Usually the saving to a community in soap alone, will pay all costs of softening. Not only does hard water destroy soap, but even when an excess of soap and alkali is used, it is not entirely suitable for laundry purposes, on account of the precipitation of insoluble calcium, magnesium, and iron salts on the fibers of washed fabrics. When these deposits dry, they turn yellow or even a darker color, particularly if the water also contains iron, organic matter, or manganese. In many communities supplied with hard water the inhabitants are forced to maintain cisterns and double-plumbing systems, and the location

Table 243. Troubles Due to Bad Boiler Feed Water

Trouble	Cause	Remedy
Incrustation	Suspended matter	Filtration: Blowing off
	Soluble salts	Blowing off
	Calcium, magnesium and iron carbonates	Softening by heat or chemical treatment
	Organic matter	Coagulation and filtration
	Calcium, magnesium and other sulfates	Sodium carbonate or hydrate, barium carbonate*
Corrosion	Organic matter	Coagulation and filtration
	Grease, Sugars	Treatment with lime and filtration
	Magnesium, chloride and sulfate	Sodium carbonate
	Acids	Alkalies
	Carbon dioxide and oxygen	Lime, sodium hydrate, heat, aeration
	Electrolysis	Zinc plates
Foaming and priming	Sewage and other pollution	Coagulation and filtration
	Alkalies	Heating feed water
	Excessive sodium carbonate	Barium chloride

* Scale formation due to sulfates (anhydrite) which takes place on the heating surfaces of boilers, may be prevented by maintaining, through additions of sodium carbonate, the carbonate content (solubility product) in excess of the sulfate content. Hall has devised a system for these additions.[4]

† If soap is 5 cts. per lb; soap used by ordinary family averages about 20 cts. per lb.

in them of many industries (dyeing, bleaching, tanning, paper making, ice making, etc.) is impracticable.

Use in Steam Boilers. Slightly alkaline surface waters of low color* are unexcelled for boiler purposes. Hard waters cause the formation of deposits or incrustations which result in loss of heat or explosions. Hard and acid waters are apt to cause corrosion.

Foaming is the action of a boiler when steam bubbles over the surface of the water to such extent that steam space and dome are filled, and siphoning action is started which causes water to be carried over with the steam. Under these conditions steam loses much of its expansive properties. Foaming is due to suspended matter[5] in waters whose concentrations of alkaline salts are high. Clear waters which contain as much as 2500 p.p.m. of sodium carbonate may generally be used without danger of foaming, while as little as 50 p.p.m. may cause foaming in presence of considerable suspended matter.

Priming is the explosive evolution of steam from a heating surface which throws water suddenly, in large volumes, into the steam space.

Classification.—Although it is difficult to draw the line between good and bad waters, the Committee of the American Railway Engineering & Maintenance-of-Way Assn.[6] has given an approximate classification of waters for boiler use:

Table 244. Classification of Boiler Waters According to Incrusting and Corroding Constituents

Concentration, p.p.m.	Concentration g. p. g.	Classification
0 to 70	0 to 4	Very good
70 to 150	4 to 9	Good
150 to 250	9 to 15	Fair
250 to 400	15 to 24	Bad
400 and above	Over 24	Very bad

SOFTENING PROCESSES

Economy. Softening would be more generally adopted were its economies better understood. Before use in steam boilers, all waters containing more than 250 p.p.m. of incrusting solids should be treated, and waters containing more than 150 parts should be treated if 50 parts of the same consist of sulfates.† While much may be done to prevent scale formation in boilers by internal treatment, it is better to soften the feed water, or, where practicable, the municipal supply from which the feed water is taken.

Processes. Two general processes are used: the lime-soda and the zeolyte processes. The former is sometimes modified for treating industrial waters high in sulfates by substituting barium for sodium carbonate. The lime-soda and zeolyte processes may be used in combination.

The lime-soda process[8] depends upon adding enough quicklime (CaO) to absorb the free carbon dioxide, change the calcium bicarbonate‡ to carbonate, the magnesium bicarbonate to hydrate, and also in adding enough soda

* Less than 50 p.p.m.

† Waters containing excessive amounts of alkaline carbonates may require distillation, using multiple effect apparatus. See cost analyses in Blast Furnace & Steam Plant, November, 1921, p. 672.

‡ For the terms total, bicarbonate, normal, and caustic alkalinity, see pp. 598 and 680.

ash (Na_2CO_3) to change the calcium sulfate to carbonate and the magnesium sulfate to hydrate. The reactions are as follows:

1. When lime is added to a hard water containg magnesium:

$$CaCO_3.CO_2 + Ca(OH)_2 = 2CaCO_3 + H_2O$$
$$MgCO_3.CO_2 + Ca(OH)_2 = MgCO_3 + CaCO_3 + H_2O$$
$$MgCO_3 + Ca(OH)_2 = Mg(OH)_2 + CaCO_3$$
$$CO_2 + Ca(OH)_2 = CaCO_3 + H_2O$$
$$MgSO_4 + Ca(OH)_2 = Mg(OH)_2 + CaSO_4$$

2. When soda ash is added to a similar water:

$$MgSO_4 + Na_2CO_3Ca(OH)_2 = CaCO_3 + Mg(OH)_2 + Na_2SO_4$$
$$CaSO_4 + Na_2CO_3 = CaCO_3 + Na_2SO_4$$

Quantities of Chemicals* may be computed by several methods.

(1) Add sufficient lime[9] to provide hydroxyl (OH) to combine with the iron, aluminum, magnesium, bicarbonate, hydrogen ions, and carbon dioxide. If the bicarbonate ion (CO_3) in the water plus that formed by change of the bicarbonate ion and carbon dioxide be not sufficient to precipitate the calcium, present in the water and added as lime, an additional quantity of CO_3 must be provided by the addition of soda ash in order that all the calcium may be precipitated. This latter consideration determines the quantity of soda ash to be added. The formula is as follows:†

$$C = 1.12\ Fe + 3.46\ Al + 2.56\ Mg + 30.96\ H + 0.51\ HCO_3 + 1.42\ CO_2$$
$$D = 2.00\ Fe + 6.18\ Al + 2.78\ Ca + 4.58\ Mg + 55.44\ H - 1.86\ CO_3 - 0.92\ HCO_3.$$

In the formula, CO_2 = carbon dioxide and H = free acid expressed in terms of equivalent hydrogen. C = the quantity of 90 per cent. lime and D the quantity of 95 per cent. soda ash in p.p.m. required to soften the water.

(2) The quantity of chemicals may be determined directly by experiment according to the method of P. Drawe.[10] 200 c.c. of the cold water to be softened are mixed with 50 c.c. of saturated lime water of known strength in a 250-c.c. volumetric flask and heated to boiling. The strength of the lime water must be determined for each series of tests. After cooling, the flask is filled to the mark with water, the contents are mixed, and 200 c.c. are filtered through a dry filter and titrated in a porcelain dish with $\frac{N}{10}$‡ HCl with methyl-orange as an indicator.

If "b" c.c. HCl are necessary for this purpose, and "a" c.c. $\frac{N}{10}$‡ CaO were contained in the 50 c.c. lime water, the milligrams of lime per liter required to soften the water tested may be computed by the formula: $(4a - 5b)3.51$ CaO. Then add to the neutralized solution in the porcelain dish 20 c.c. $\frac{N}{10}Na_2CO_3$ and heat to the beginning of boiling. The contents of the dish are washed with water free from CO_2 into a 250 c.c. flask, cooled, made up to the mark,

* For table of chemical formulas see p. 676.

† The elements are determined by quantitative analysis and multiplied by the various factors in the formula. The quantities so determined are then summed.

‡ See footnote, p. 596.

mixed, and filtered. 200 c.c. of the filtrate are measured off and the excess of alkali titrated with $\frac{N}{10}$HCl. The number of c c. used is designated by *c*. Then the number of milligrams of soda per liter required to soften the water treated may be computed by the following formula:

$$(20 - b - \tfrac{5}{4}c)33.13\ Na_2CO_3$$

The results by these formulas are for pure chemicals. For 90 per cent. lime and 95 per cent soda corrections should be made. In order to get good results by this method the most careful work and the exact preparation of normal solutions, etc., are necessary. When the hardness is over 350 p p.m., 100 c.c. of water should be used for the determination For alkaline waters *b* in the formulas is to be diminished by the number *d*, which equals $\frac{8}{5}$ (carbonate hardness —total hardness). The correctness of the above amounts for the desired effect can be proved only by a practical experiment For this purpose Drawe takes 1 L. of water, adds the calculated amounts of dry lime and soda and heats the whole to 70°C. Water treated in this way as a rule has a slight soda alkalinity and a hardness of less than 18 p p m.

Computed directly from the free carbon dioxide, the half-bound carbon dioxide (44 per cent of the alkalinity), the mineral acid hardness (incrustants), and the total magnesium, the quantities are as follows:

a Lime required

Constituent p p m	Factor	Quantity CaO
Free and half bound carbon dioxide	1 273	as p p.m
	10 618	as lbs per mg.
	0 074	as g p g
Total magnesium	2 302	as p p m
	19 21	as lbs per mg
	0 136	as g p g

b. Soda ash required

Constituent p p m	Factor	Quantity Na_2CO_3
Incrustants or mineral acid hardness	1 060	as p p.m.
	8 846	as lbs. per mg.
	0 619	as g.p g.

The above figures are for pure chemicals. Corrections should be made for commercial chemicals or when calcium hydrate is used in place of lime. See p. 655 for descriptions of, and specifications for, softening chemicals.

Collins[11] has devised a graphical scheme for performing the calculations incident to the softening process, using arbitrary figures obtained by dividing the atomic weight by its valence.

Caustic soda* is also sometimes used for softening waters containing "permanent" or "mineral acid" hardness. It costs from 2 to 2.5 times as much as

* For volumes and weights see p. 678.

soda ash and is useful only where small quantities are required, where the presence of carbonates is undesirable, or where a speedy reaction is desired. Caustic soda is sometimes added to hot boiler-feed waters. It reacts very quickly, and the precipitate formed may be removed by a filter press or pressure filter, or to a great extent in a feed-water heater. Caustic soda is furnished in drums. The best caustic soda contains about 76 per cent. of Na_2O.

Barium carbonate is sometimes used to remove calcium sulfate ($CaSO_4$) from water. Barium carbonate has the formula $BaCO_3$, and is practically insoluble in water free from CO_2. In the presence of water containing $CaSO_4$, however, $BaCO_3$ reacts according to the following formula:

$$BaCO_3 + CaSO_4 = CaCO_3 + BaSO_4$$

Barium carbonate is a more expensive softening chemical than sodium carbonate, but the latter has the disadvantage of producing sodium sulfate (Na_2SO_4) as a by-product, which has a tendency to cause foaming in steam boilers unless the boilers be blown off frequently, while the insoluble $CaCO_3$ and $BaSO_4$ are precipitated during softening. Soluble barium salts are poisonous.

Coagulation and Precipitation. Many of the compounds formed by the reactions are precipitated in the colloidal condition and coagulated slowly. This is particularly true of calcium carbonate, the chief constituent of many waters. Where magnesium hydrate is precipitated, coagulation is rapid. Coagulation and precipitation are greatly assisted by agitation. For this purpose, mixing channels or stirring devices should be employed to hasten the reactions.[12]

Effect of Excess of Lime. Addition of enough lime to remove the free CO_2 has a marked bactericidal effect, as shown by Hoover at Columbus, where the softening plant removes nearly all the bacteria; also by Houston ("Studies in Water Supply," 1913–1918).

Notes Regarding the Lime-soda Process. *Time and Temperature.* During the years 1913–1924, 12,000 determinations at one municipal softening works indicate the period of reaction between the softening chemicals, calcium and magnesium, is short; 90 per cent. of the reaction occurs in the mixing tanks which have a 2-hr. storage period, 3 per cent. in basins storing 15 hr., and 7 per cent. in passage through the filters. The figures point to better mixing and smaller subsiding basins and indicate that colloidal precipitates are formed by the reaction and are crystallized by agitation.

Stirring. The period recommended is 20 to 30 min., velocity from 0.6 to 1.5 ft. per sec. Mechanical agitator tanks are preferred to baffles in basins.

Return of Sludge. The continuous return of 50 per cent. of the sludge separated at the bottom of the clarifiers to the water as it enters the mixing tank, increases the efficiency of floc formation, causes better crystallization of the floc formed, and saves chemicals.

Overtreatment. Non-carbonate hardness and magnesium require overtreatment and neutralization of the excess lime with soda ash. This is in addition to the amount necessary to combine with the non-carbonate hardness and leaves an excess of sodium hydroxide in the treated water. By excessive treatment it is possible to reduce the hardness from 1000 p.p.m. or more to less than 50 p.p.m.

Split Treatment. This consists of overtreating with lime and soda ash and then neutralizing the excess with raw water. It is more effective for a given quantity of chemicals than the ordinary method, but not as effective as overtreatment.

Carbonation. Without it sand grains in filters increase in size, and sand must be replaced every few years. Gas-producer is most efficient source for CO_2; for use at Oklahoma City, see *E. N. R.*, Aug. 7, 1924, p. 216.

Cost. A softening plant costs about $40,000 per Mgd. on the average.

The Zeolyte Process.* "Permutit" ("permutare," to exchange) is the trade name of an artificial zeolyte introduced by L. Gans, of Berlin, in 1906. Sodium permutit is made commercially by fusing quartz or feldspar, kaolin, and sodium carbonate in an electric furnace and afterward treating the product with water. It is marketed in granular form having particles from 0.5 to 2.5 mm. in diameter. The reaction† between hard water and zeolyte may be illustrated by the following:

$$\begin{bmatrix} CaCO_3 \\ MgCO_3 \\ CaSO_4 \\ CaCl \\ \text{Etc.} \end{bmatrix} + \underset{\text{(Zeolyte)}}{NaAl(SiO_3)_2} = \begin{bmatrix} Ca \\ (Mg) \end{bmatrix} Al(SiO_3)_2 + \begin{bmatrix} Na_2CO_2 \\ Na_2SO_4 \\ NaCl \\ \text{Etc.} \end{bmatrix}$$

Hardness may be decreased to zero, but soluble sodium salts will be increased proportionally. This proceeds until equilibrium is reached. The zeolyte is then regenerated by reversing the reaction by treatment with strong brine (NaCl) which exchanges its sodium for the calcium and magnesium removed from the water and stored in the zeolyte. Artificial sodium permutit is slowly decomposed by free carbonic acid. To obviate this, the water may have to be passed through a layer of marble. This adds to the hardness to be removed by the permutit. The device for using zeolyte resembles a pressure mechanical filter with zeolyte in place of sand. More than 5 per cent. of the zeolyte per annum disintegrates and washes away. For a continuous supply duplicate filters are required, one being in service while the other is being regenerated. Compared with the lime-soda process, softening by zeolyte is expensive, but for dyeing and other purposes requiring perfectly soft water, it is useful.‡ Where hardness is due to sulfates and salt is cheap, it may compete with the older process. The installation must be large enough to accommodate overload. Zeolyte plants operate at rates as high as 6 gal. per sq. ft. per min., and therefore require less room than a plant for the lime-soda process. Where the hardness is excessive, zeolyte may be used as a supplement to the lime process. In any case, care must be taken to remove the suspended matter, especially hydrates, which would accumulate on the surface of the permutit and prevent the desired interchange of bases.

* The Permutit Company, N. Y. C.

† Technically, an exchange.

‡ O'Callaghan[18] states that Permutit process alone is not effective where the hardness exceeds 40 g.p. gal.

Other artificial Zeolytes* are: Verdite;† Crystalite;† Doucil;[13]‡ Decalso§ (formed by precipitation); and Borromite,|| the latter made by heating glauconite, "green sand," by a patented process.

Experiments by the Ohio Valley Water Co.¶ (1923–1924) showed that 100 cu. ft. of "Borromite"** would soften 22,000 gal. of water having an average hardness of 190 p.p.m., between regenerations, at the rate of about 1 g.p.m. per cu. ft. of "Borromite." Then 5.43 lb. of salt were required to regenerate the Borromite for each 1000 gal. softened to zero. Borromite offers more resistance to passage of water than sand of same effective size. The plant installed by the company (1925) consists of four units, each having an area of 210 sq. ft. and a depth of 40 in.; average capacity 4 mgd; maximal 6 mgd. This material removes manganese from water as well as softens.

Boiler Compounds. While a steam boiler is not well adapted for the precipitation of chemicals, boiler compounds have their legitimate use, especially

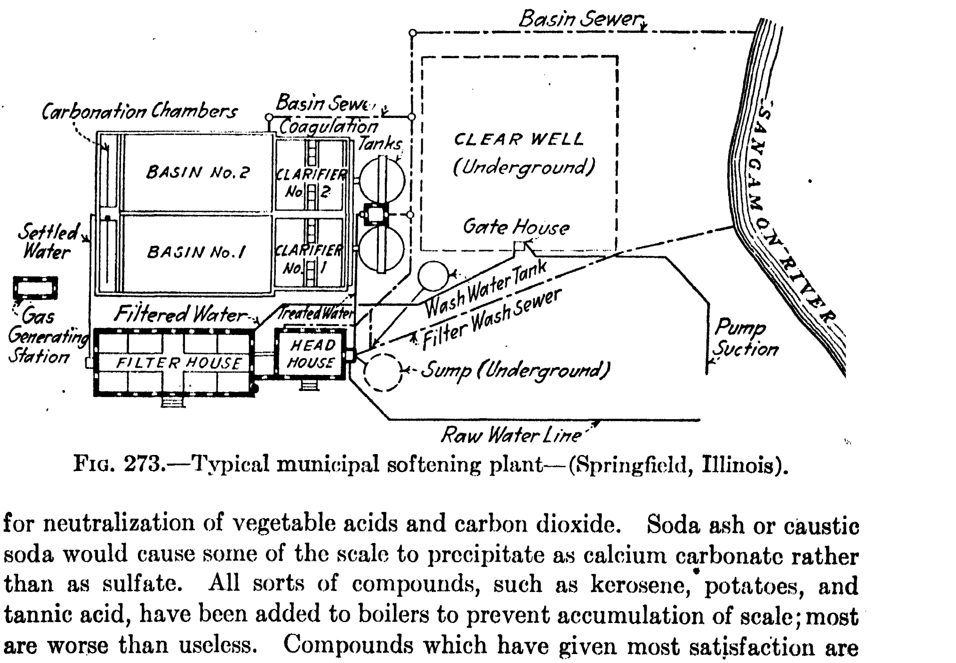

FIG. 273.—Typical municipal softening plant—(Springfield, Illinois).

for neutralization of vegetable acids and carbon dioxide. Soda ash or caustic soda would cause some of the scale to precipitate as calcium carbonate rather than as sulfate. All sorts of compounds, such as kerosene, potatoes, and tannic acid, have been added to boilers to prevent accumulation of scale; most are worse than useless. Compounds which have given most satisfaction are combinations of soda and tannic acid. Tannic acid has a slight action on the iron and prevents accumulation of scale. A compound consisting of 1 gal. of hemlock extract with 2 gal. of water and 3 lb. of soda ash may be used with many waters. Trisodium phosphate (Na_3PO_4) is used with excellent results in many boilers fed with acid waters. Metal treatment by introducing zinc,

* "An Impartial Discussion on the Respective Merits and Disadvantages of Methods of Water Softening by Filtration through Zeolytes, by Chemical Precipitation and by Rectification with Boiler Compounds," by W. M. Taylor. *Chem. Met. Eng.*, Jan. 19, 1921, p. 123. Also Behrman, "Lime-soda and Zeolyte Water Softening," *J. A. W. W. A.* 10, p. 627 (1923).

† International Filter Co., N. Y.

‡ American Doucil Co., Phila.

§ American Water Softener Co., Phila.

|| The Wayne Tank & Pump Co., Fort Wayne, Ind.

¶ McKee's Rocks, Pa. See Beech, *J. A. W. W. A.*, Vol., 15, 1926, p. 227.

** SiO_2 = 50.3 per cent.; Fe = 11.0 per cent.; Al = 1.33 per cent.; Ca = 1.86 per cent.; Mg = 2.23 per cent.; Na = 4.17 per cent.

graphite, "Perolin," and similar substances which penetrate the scale and form insulating coatings on the metal has been tried with varying success.[14]*

Types of Water Softening Plants.† Softening plants are either continuous or intermittent, and designed to operate with either hot or cold water. In the intermittent system, chemicals are added to water contained in a basin; the whole is mixed, allowed to subside, and the treated water is decanted. In the continuous system, chemicals are added proportionally to the flow of water and the treated water is passed through subsiding basin and filter. Figure 274 illustrates a softening system, operating on the intermittent plan; Fig. 275 shows one operating on the continuous plan. In the example of latter system illustrated, lime water is prepared in a lime-water tank, or saturator, and

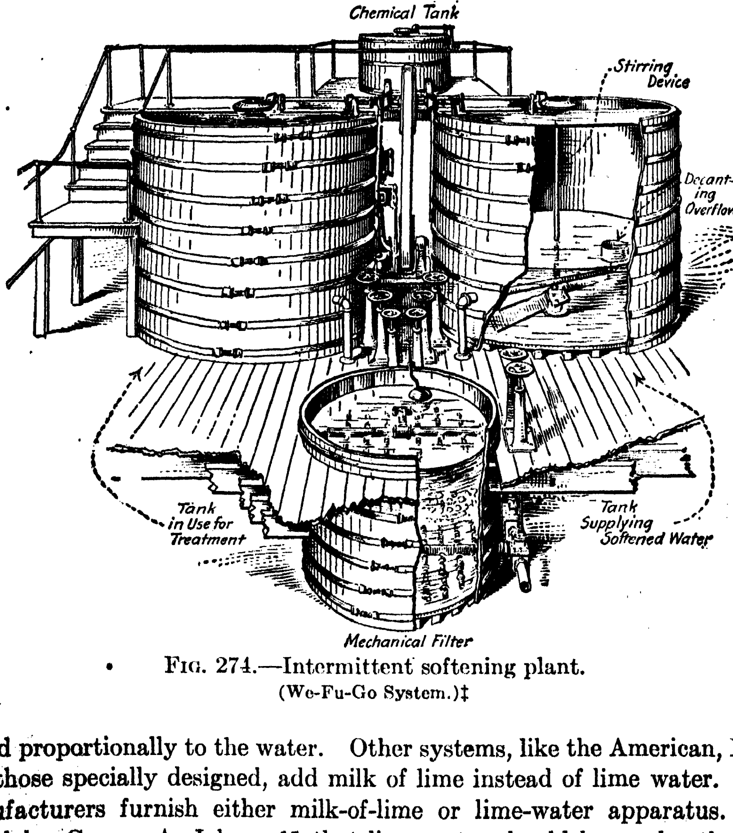

FIG. 274.—Intermittent softening plant.
(We-Fu-Go System.)‡

added proportionally to the water. Other systems, like the American, Booth, and those specially designed, add milk of lime instead of lime water. Some manufacturers furnish either milk-of-lime or lime-water apparatus. It is stated by George A. Johnson[15] that lime water should be used rather than milk of lime, because with the latter the suspended particles of calcium hydrate become coated over with $CaCO_3$ and are rendered inactive. Lime water was first used at Columbus, Ohio. Later, milk of lime was substituted. At St. Louis weighed quantities of lime are added to the water at definite intervals of time. The apparatus required to add milk of lime is considerably less costly and cumbersome than that required for lime water.

* See French in *I. & E. C.*, December, 1923, p. 1239.
† For data on municipal plants, see p. 692.
‡ W. B. Scaife and Sons, Pittsburgh.

We-Fu-Go system* (Fig. 274), involves a chemical tank in which the reagents are mixed in the required proportions, a pair of mixing tanks, and a mechanical filter. In the mixing tanks the chemical solutions are mechanically stirred with the water, a tank is left quiescent during the settling period, while the second tank is stirred—an intermittent process. The effluent from the settling tanks is passed through the mechanical filter.

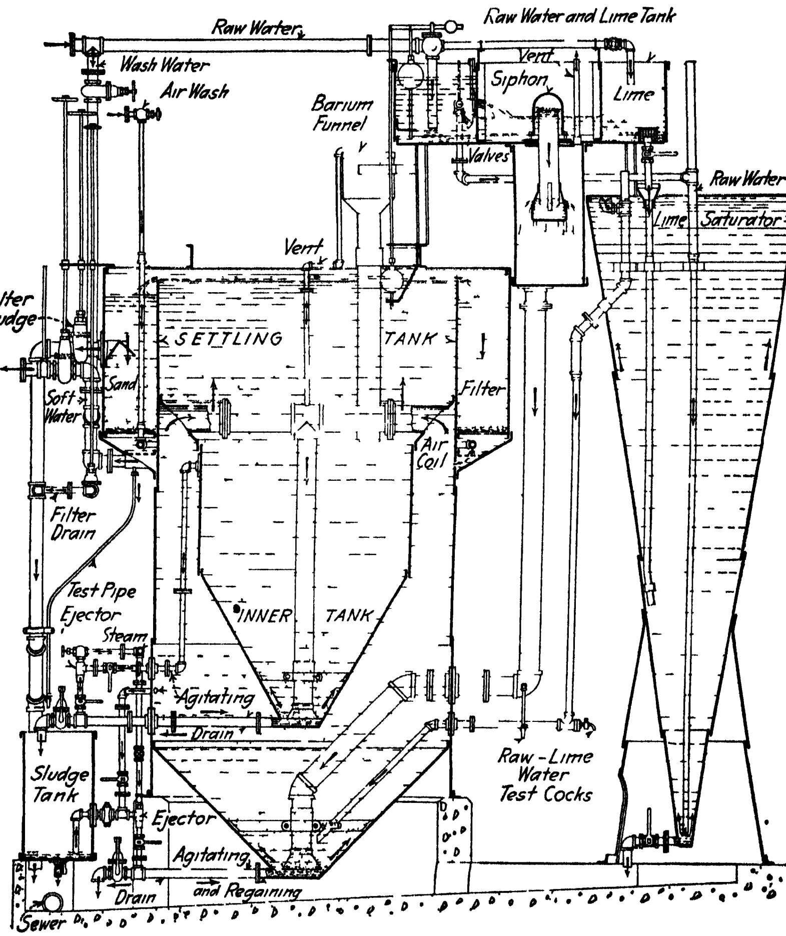

FIG 275 —Reisert lime-barium water softener.
(Reisert Automatic Water Purifying Co , New York)

For the barium process,† the apparatus illustrated by Fig. 275 is used. In order to effect the dissolving of the $BaCO_3$, it is necessary to keep it agitated. This is accomplished by the intermittent discharge of the water through a siphon. In general, the plant resembles the lime-soda apparatus. There is a chemical or distributing tank above the main settling tank in which

* Typical installation is described in Blast Furnace & Steel Plant, February, 1921, p. 128.
† Plant for Chino Copper Co at Hurley, N. M., is described in *C. & M. E.*, Nov. 12, 1919, p. 629.

the lime is slaked and run to the raw water, and which also contains the raw-water chamber and the raw-water feeding device. The lime when slaked is run into the bottom of the lime saturator and thereafter a carefully regulated quantity of raw water is passed through the lime and returns through an overflow pipe at the top of the saturator to mix with the raw water in the outer settling tank in the form of a clear, saturated, lime water of constant strength.

The barium is introduced, through the barium funnel, into the inner tank, where the raw water is brought in contact with the barium. $CaCO_3$ and $Mg(OH)_2$ are precipitated in the outer compartment, and therefore do not prevent the proper mixing of the raw water with the barium carbonate. The settling tank contains a down-take pipe extending very near to the cone-shaped bottom of the compartment. This down-take pipe connects at its

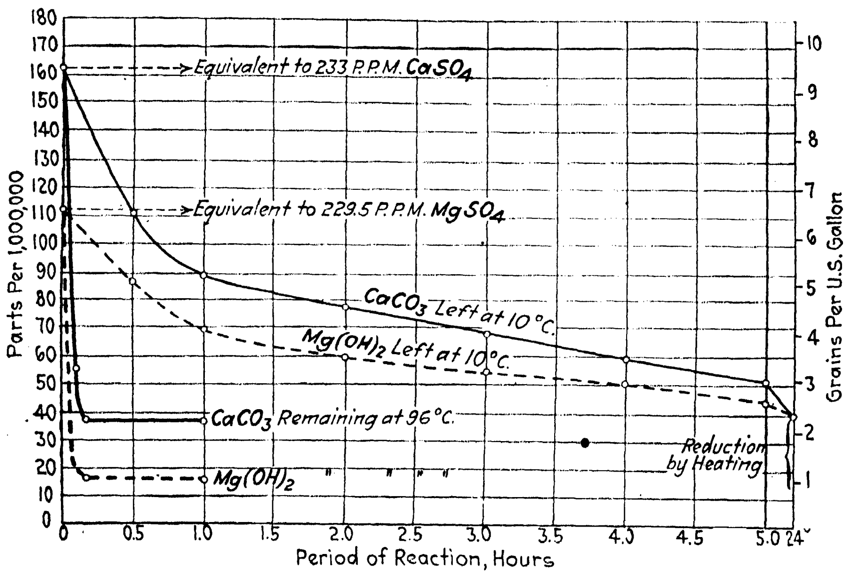

FIG. 276.—Value of heat in water softening.
(H. S. B. W.—Cochrane Corp.)

upper end with a chamber in which is located an automatic siphon so arranged that the raw water is fed to the settling tank intermittently in previously calculated quantities at a time. The effect of this intermittent feeding of the raw water is to produce a regularly recurring pulsation in the water in the settling tank which has the effect of keeping the heavy barium carbonate, which is in a very finely divided state, in continual agitation and brings the barium carbonate into intimate contact with every portion of the raw water. The barium carbonate, while insoluble in pure water, is soluble in selenitic* waters in the proportion that the sulfate radical is present in the water. A selenitic raw water will, therefore, dissolve barium carbonate until its sulfate contents are satisfied. The reaction which takes place then between the barium

* Containing calcium sulfate.

carbonate and the calcium sulfate produces barium sulfate and lime carbonate, which are both insoluble and are precipitated. This precipitate gradually drops out of the water in its upward course in the settling tank, but to insure the delivery of an absolutely clean water a filter is embodied in the settling tank, which effectually intercepts any precipitate which may still remain in the water. In some plants, in order to prevent the precipitated calcium carbonate from interfering with the action of the barium carbonate, the temporary hardness is removed before the barium carbonate treatment.

Hot Process. The value of heat in water softening as illustrated by the Sorge-Cochrane Hot-process Water Softener is shown in Fig. 276.

Municipal plants usually operate continuously. Figure 273 shows the new plant at Springfield, Ill., which is typical. Other plants built since 1920, include Oklahoma City, South Pittsburgh, Newark,[19] and Piqua, Ohio. At South Pittsburgh, recarbonation is effected by adding a sufficient amount of raw water, high in carbon dioxide. At Newark,[16] split treatment, mixing tanks, and Dorr clarifiers are used, in addition to two subsiding basins storing 8 hr. flow. The Dorr clarifier removes 98 per cent. of the precipitate, and smaller basins might have been used. 100 lb. coke per mg. are required to maintain an excess of 1 p.p.m. CO_2 in water applied to filters. Scrubbed gas is applied to water through perforated pipes laid in weak, coarse concrete.

Filters are usually required for softening plants and differ but little from typical rapid filters.

Results of Operation. The following results of operation are typical.

Table 245. Analysis of Water before and after Softening, Port Tampa, Fla. (1914)

Grains per U. S. Gal.

	Before	After
Calcium carbonate	16.62	0.31
Calcium sulfate	0.35	....
Calcium chloride	13.10	....
Calcium hydroxide	....	0.78
Magnesium chloride	4.78	....
Magnesium hydroxide	....	0.24
Iron oxide / Alumina	0.33	0.12
Silica	1.66	0.93
Suspended matter	0.25	....
Incrusting solids	37.09	2.38
Sodium carbonate	....	2.15
Sodium sulfate	....	0.38
Sodium chloride	28.63	48.30
Non-incrusting solids	28.63	50.83
Free carbon dioxide	0.66	....
Half-bound carbon dioxide	7.31	....
Volatile matter	7.97	....

Table 246. Comparative and Average Results of Operation at Columbus, O.

Hoover

Year	Volume of water softened and purified, gallons per 24 hours	Total hardness parts per million		
		River water	Filtered water	Per cent. removed
1909	14 3	253	93	63
1910	15.5	270	85	68
1911	15.6	245	84	61
1912	17.5	222	79	67
1913	18.3	271	88	67
1914	18.4	297	79	73
1915	17.7	279	88	69
1916	19.8	279	110	61
1917	21.6	278	125	55
1918	23.8	306	125	59
1919	22 7	278	106	62
1920	23.6	266	109	59
1921	21 4	269	100	63
1922	22 0	278	101	64
1923	23 9	265	95	64

Table 247. Cost of Operation and Maintenance of Water Softening and Purification Works, Columbus, Ohio

Per Million Gallons Treated. Hoover

Year	Entire plant	Chemicals	Entire plant less chemicals
1909	$18.35	$9 75	$8.60
1910	17.85	7 80	10.05
1911	17.31	10 60	6.71
1912	14 58	8.90	5.68
1913	16 08	10 50	5.58
1914	17.46	12 20	5.26
1915	15 34	9.80	5.54
1916	27 75	23.20	4.55
1917	25 50	21.08	4.42
1918	36.81	30.78	6.03
1919	27.44	21.64	5.80
1920	26.33	19 45	6.88
1921	37.32	29.04	8.28
1922	29.22	21.22	8.00
Average	22.91	16.38	6.53

Table 248. Results of Analyses of Water from Water Treatment Plant at Defiance, Ohio.[17]

Parts per Million

Constituents	Inlet, settling basin		Outlet, settling basin		Outlet, carbonating chamber—carbonated water		Outlet, filters—filtered water	
	Sample 8754 10-23-22	Sample 9421 4-10-23	Sample 8755 10-23-22	Sample 9422 4-10-23	Sample 8756 10-23-22	Sample 9423 4-10-23	Sample 8757 10-23-22	Sample 9424 4-10-23
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Turbidity	48	375	12	13	1	6	0	0
Color	24	43	6	14	6	10	9	8
Total solids	419	687	238	241	241	244	242	250
Suspended solids	65	367	19	9				
Solids in solution	354	320	219	232				
Iron							0.05	0.05
Total hardness as calcium carbonate ($CaCO_3$)	247	218	99	117	120	126	121	129
Lime (Ca) expressed as $CaCO_3$	153	165	67	105	84	111	79	106
Magnesium (Mg) expressed as $CaCO_3$ (by difference)	94	53	32	12	36	15	18	23
Caustic alkalinity expressed as $CaCO_3$	0	0	20	2	0	0	0	0
Normal carbonate alkalinity $CaCO_3$	14	0	16	32	0	0	0	0
Bicarbonate alkalinity $Ca(HCO_3)_2$ expressed as $CaCO_3$	172	163	0	0	50	33	47	35
Free carbonic acid (CO_2)	0	5.3	0	0	0	5.1	1.3	0
pH	8.1	7.7	9.8	9.5	7.85	7.1	7.65	7.8
Chlorides (Cl)	16.9	5.3	16.3	6.5	16.0	6.2	16.0	7.7
Sulfates (SO_3)	66	56	77	73	79	71	75	76

Table 249. Average Results for 1922, Defiance, Ohio[17]

	Raw water, p p m	Filtered water, p p m
Turbidity	200	0
Color	40	8
Total hardness	230	125
Alkalinity	170	52
Incrustants	60	73
Calcium hardness	172	70
Magnesium hardness	58	55
Bacteria per cubic centimeter at 37°C	400	

Table 250. Comparison of Lime-soda and Lime-barium Processes[18]

	Raw water, p p m	Lime-soda process, p p m	Lime-barium process, p p m
Total residue	351.5	209.6	61.4
Mineral	286.9		
CO_2	73.5		
Silica	7.2		
Oxide of iron and alumina	1.0		
Lime	130.5	7.8	8.0
Magnesia	21.4	10.3	9.2
Soda	5.4	77.5	5.4
Sulfuric anhydride (SO_3)	102.4	102.4	4.2
Nitric acid	Trace		
Chlorine	Trace		
Total hardness	285.5	39.3	37.5
Permanent	167.7		
Temporary	117.8		

Table 251. Results by Lime-barium Process, Reisert Apparatus, Ray Consolidated Copper Co., Hayden, Ariz.

Total hardness	Parts per million	
	Raw water	Treated water
Total hardness	488	25.7
Temporary hardness	240	....
Permanent hardness	248	....

Table 252. Typical Results of Softening Feed Water by the Hot Process (Sorge-Cochrane) (p.p.m.)

	Raw water	Softened water
Calcium sulfate	96.5	None
Calcium carbonate	115.9	16.1
Magnesium sulfate	14.7	None
Magnesium chloride	158.0	None
Magnesium carbonate	....	3.9
Silica	9.9	9.9
Iron and aluminum oxides	3.6	Trace
Total incrusting solids	398.6	29.9
Volatile and organic	124.6	21.9
Sodium carbonate	....	16.9
Sodium sulfate	....	100.8
Sodium chloride	54.92	988.9
Total non-incrusting solids	1064.7	1127.7
Free carbon dioxide	8.7	....

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CHAPTER 32

MISCELLANEOUS PURIFICATION PROCESSES

Distillation. Whenever a water of highest purity is an imperative requirement, distillation is the only method whereby it can be produced. The cost of distillation is so high, as compared with the other methods of purification, that its commercial use is limited.[1]

Degasification. Studies by Speller[2] show that removal of oxygen from water will lessen its corroding power.* Degasification (or deactivation) may be accomplished by mechanical methods, chemical methods, or a combination of both. The principal mechanical methods are: agitation of the water, temperature, vacuum—used either separately or in combination. The principal chemical method consists in placing the heated water in a tank filled with iron or high manganese-steel scrap, which effect the absorption of oxygen. The Kestner degasser is used in Europe; deäerators of the Elliot and other types based on the Speller process are used in this country.[3]

Collection and Purification of Rain-water. Along the Gulf shores of the United States, where the annual rainfall averages over 60 in., rain-water supplies are the rule in all but the largest cities. Rain-water should be collected from clean surfaces; the first runnings should be rejected. It should be stored in wooden or masonry cisterns, never in lead-lined or unprotected steel tanks. Cistern overflows should not connect with sewers. Rain-water usually has a mawkish taste which may be overcome by filtration and aeration. Rain-water filters should be designed with care; water should be stored before filtration to insure a low rate of filtration. For supplying 400 gal. daily for 4 months yearly, to a residence at Little Compton, R. I., from a catchment area (2500 sq. ft.) in the form of a shingled roof, there were required a 10,000-gal. rain-water basin to utilize run-off during heavy showers, a slow sand filter 9 sq. ft. in area, and a filtered-water basin holding 40,000 gal. Cost, in 1915, was $1500, exclusive of pumps, motors, and piping. Rain-water filters should have at least a 3-ft. depth of sand having an effective size of from 0.15 to 0.30 mm. Maximum rate of filtration, 6 mgad. Filters should be examined frequently and scraped when necessary. Provision should be made for aerating the water filtered. Sediment in cisterns should be removed frequently.

Scrubbers, preliminary filters, and contact baffles are filters of coarse material to take the place of, or add to the efficiency of, subsiding or coagulating basins. Figure 277 shows the contact baffles installed with the subsiding basins at Pittsburgh. These baffles are units each about 40 by 60 ft., and designed for a filtration rate of 67 mgad. The contact material is gravel from 0.5 to 1 in. in diam.; depth, 8 ft. The baffles are cleaned by shutting down

* See also pp. 431, 661 and 810.

one unit at a time, and scouring out the accumulated matter with the streams from a high-pressure fire-hose. Preliminary filters of this type owe their

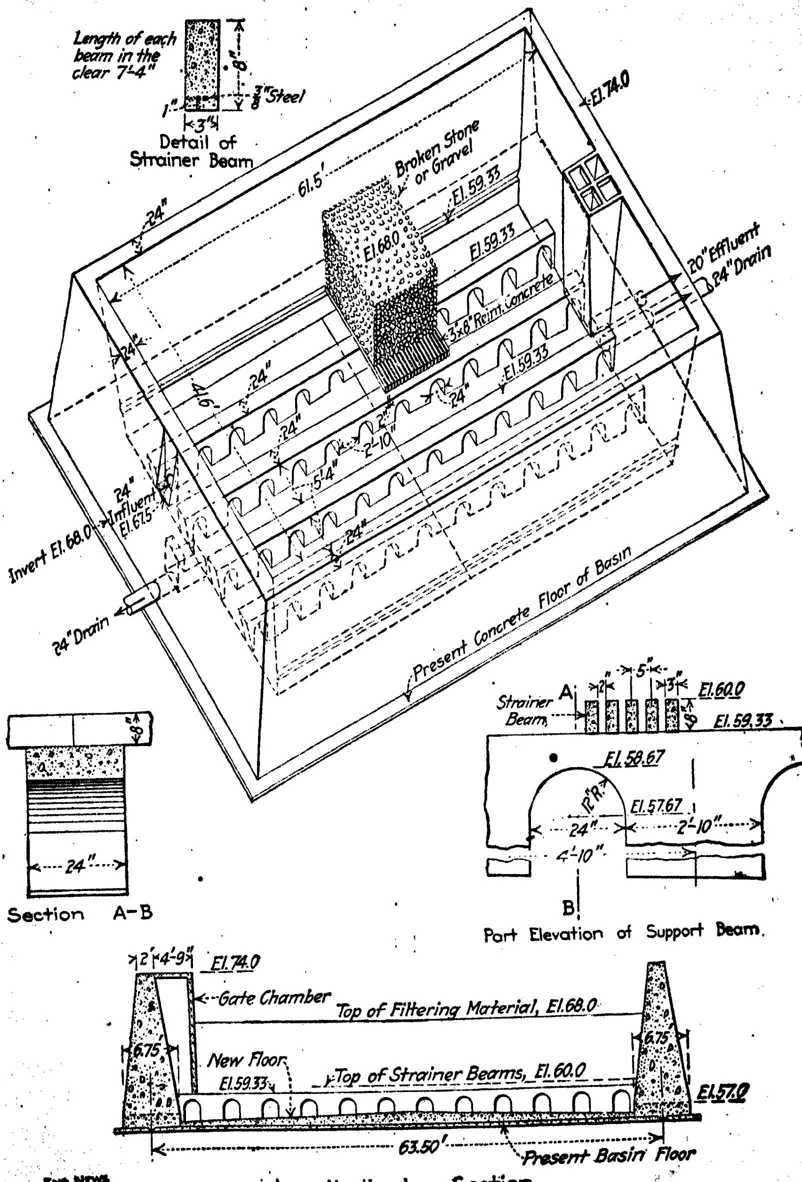

Fig. 277.—Isometric view of typical unit and some details of contact filters, Pittsburgh, Pa.[4]

efficiency in part to plain subsidence, and in part to the removal of colloidal matter by contact. For subsidence alone, basins* are usually more economi-

* See Chap. 28.

cal. Contact is of great assistance in coagulation, and in replacing CO_2 and H_2S by oxygen. Preliminary coarse filters may be designed efficient for removing suspended matter, even quite fine particles, but are difficult to clean unless of the rapid filter type. Usually, preliminary filters, excepting rapid sand filters, filled with material of a diameter smaller than 15 mm., are not practical unless the water contains iron or manganese or both, and little suspended matter.

Intermittent filters are demanded where the water contains so much organic matter that the treatment must approach that for sewage. They are especially necessary for treating odoriferous waters such as exist in the tropics and in certain shallow reservoirs in the temperate zone. Such waters are difficult to purify, either by slow or mechanical filters of the ordinary type.

Double filters are also used. In Europe double filtration without coagulation at fairly rapid rates is preferred to rapid filtration with chemicals.

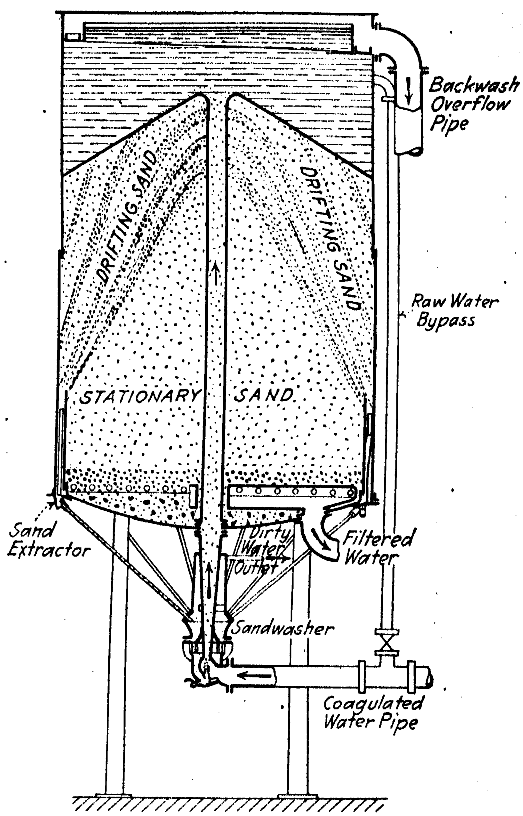

FIG. 278.—Typical unit of drifting-sand filter. (There are 30 of these units in each of the 10 filters of the Toronto plant.)

Drifting sand filters, a modification of rapid sand filters, have been used in Toronto[6] since 1916. Coagulant is introduced as water goes to filters. By causing the upper part of the sand bed to drift across the path of the raw influent water, most impurities are swept out, together with a part of the drifting sand, the latter being washed and returned by the constant circulation of the water and sand. The stationary lower portion of the sand bed takes out the remaining impurities. As often as conditions demand, the bed is washed by reversed flow of filtered water. The coagulated raw water enters the filter partly by a standpipe at the center of the unit (Fig. 278), passing up through a separator or sand washer at the bottom, and discharging above the sand at the top, and partly through a bypass. Within the sand washer, the raw-water pipe is constricted like the tube of a Venturi meter, and the drifting sand collected and washed in the separator is inducted into the raw water at the throat

of the Venturi tube. This sand passes up the standpipe with the water and is delivered with it above the top of the sand already there, forming a volcano-like cone that continuously drifts away and is continuously being replaced, leaving a round-topped body of stationary sand below, whose surface is more than twice the plan area of the unit, which is an economy. The drifting sand passes down outside the stationary sand to a slot and ultimately to the sand washer. Here the sand falls to the bottom through a current of raw water and is picked up by the inductor.

The loss of head in the filters is initially 6 ft., gradually increasing to 11 ft., when the filter is back-washed. The length of run is 1 to 7 days, according to condition of raw water and amount of alum. Back-washing is effected by reversing the flow of filtered water through the bottom of the filter, the wash water coming from an elevated tank having a head of 25 lb. The wash water is 1 to 2 per cent., and in addition 2 per cent. of the water treated is lost through the sand washer.

Table 253. Toronto Drifting Sand Filter Plant Data, Toronto, Ont.

Designer	William Gore	
Plant put in operation		
Population (1920)	512,812	
Total cost, exclusive of land and wharf	$1,122,256.50	
	Value of land for reservoir	7,937.50
	Value of land	16,250.00
	Reservoir	45,000.00
	Wharf	11,592.50
		$1,203,036.50
Source of supply	Lake Ontario	
Rated capacity (gallons per acre per day)	150,000,000 Imp.	
Total cap. of subsiding or coagulating basins (gals.)	None	
Chemicals used	Alum or chlorine or both	
Manner of application of chemicals.	Alum dissolved. Density of solution varied in hydrometer chamber according to dosage. Application by Venturi rate controller proportionate to pumpage. Alum applied to suction of low lift pump.	
No. of filter units	10	
Net area of filter surface (acres)	0.4	
Depths of filtering materials (inches)	Gravel (9) Sand (10)	
Sizes of filtering material (millimeters)	Eff. size 0.35–0.4 Uniformity coef. 1.8	
Cleaning of filter system	Continuous washing of drifting sand in sand-washers. Raw wash water used 2½%. Backwashing at intervals by usual mechanical filter method Filtered wash water used: 1–1½ per cent.	
No. of coagulating or subsiding basin	None	
Control of rate of filtration	Hand regulation. Registering and indicating Venturi meters and registering loss of head gages.	

Deferrization is the process of removing iron from water. In the absence of oxygen, this is soluble, unoxidized, and usually accompanied by mineral salts, carbon dioxide, and other gases, and perhaps by organic matter or manganese, or, indeed, free sulfuric acid. Iron can be precipitated from most ground waters—those low in manganese and vegetable organic matter—by simple aeration by spraying, followed by filtration through sand or even fine gravel. Provided the water be properly treated beforehand, the kind of filter has little to do with removing iron. Simple aeration oxidizes the iron from the soluble unoxidized form to the insoluble oxidized form, from ferrous to ferric hydrate. It also effects the removal of some of the carbon dioxide. One p.p.m. of oxygen will oxidize 7 parts iron; an excess of oxygen may be obtained by slight exposure to air. Ferrous hydrate oxidizes quite rapidly to ferric hydrate. This latter is insoluble and in most cases coagulates and precipitates rapidly. Some waters contain interfering substances which prevent the precipitation of the iron, holding it in the colloidal form and making its removal by filtration difficult. *Acids, organic matter, and manganese* all interfere with the precipitation of ferric hydrate. It is not the oxidation, but the coagulation of iron, which is difficult to accomplish, as is the coagulation of similar small amounts of aluminum, 0.5 to 5 p.p.m. The reactions involved may be expressed as follows:

A. When iron exists as bicarbonate,

1. $(Fe''CO_3 + CO_2)^* + O + H_2O = Fe''(OH)_2 + 2CO_2$
   Bicarbonate + air + water = Ferrous Hydrate + Carbon Dioxide.
2. $2Fe''(OH)_2 + 2O + 2H_2O = Fe_2'''(OH)_6$ (Ferric Hydrate).

B. When iron exists as sulfate,

1. $Fe''SO_4 + XH_2O \rightleftarrows Fe''(OH)_2 + H_2SO_4 + XH_2O$
   until equilibrium is established.

Then $H_2SO_4 + (CaCO_3 \text{ or } Na_2CO_3) = CaSO_4 + CO_2$
Then $Fe''(OH)_2 + CO_2$ is hydrolyzed to $Fe_2'''(OH)_6$ as in *A*.

Pretreatment. Water must be so treated prior to filtration that not only will the iron be oxidized but the carbon dioxide will be eliminated as much as practicable. However, certain soft waters like those at Reading and Lowell,[7] Mass., and West Superior, Wis., which are from alluvial deposits or beneath marshy areas, cannot be saturated with oxygen without causing the formation of a compound of iron and organic acid which is extremely difficult to remove. The presence of a small quantity of carbonic acid seems to be necessary to prevent the formation of this compound. Certain kinds of organic matter mutually precipitate one another. Other kinds, however, interfere seriously with the deferrization process; at Merrimac, N. H., potassium permanganate had to be used to accelerate oxidation.

Removal of CO_2. Where complete removal of CO_2 is requisite for coagulation of iron, it can be accomplished either by discharging the water over a bed of coke from 2 to 10 ft. thick and at a rate of about 75 mgad, or by discharge over superimposed shallow trays filled with coke, as at Memphis. Waters like those at Lowell and Reading can be coagulated and the interference of organic matter avoided by operating the coke bed submerged instead of trickling the water over it. This increases the time of contact.

* F'' = ferrous iron (bivalent); F''' = ferric iron (trivalent).

At *Brookline, Mass.*,[8] *experiments* showed efficiency of preliminary treatment to be dependent largely upon amount of dissolved oxygen added and carbon dioxide removed. Table 254 shows the exchange of gases brought about by different methods; also the beneficial effect of age due to accumulations of iron hydrate on the surface of the filtering material.

Effect of Storage. Coagulation could be accomplished, but often at a prohibitive cost, by storing the water for 24 hr or more. The plant at Middleboro, Mass [8] (see Table 256, p 701) is typical During 1914, the average results of deferrization were those given in analysis, Table 255. The plant operated a part of each day and delivered 320,000 g p d (nearly a third of its possible capacity)

Table 254. Dissolved Oxygen* and Carbon Dioxide† at Brookline, Mass.

Date 1913	Well water		Effluent from devices for preliminary treatment							
			Spray aerator and 9 3-hour coagulating basin		Spray aerator 2-ft. coke trickler and 1-hour basin		Spray aerator, 5-ft coke trickler and 1-hour basin		Spray aerator, 10-ft. coke trickler and 1-hour basin	
	O	CO_2	O	CO_2	O	CO_2	O	CO_2	O	CO_2
Aug 7	1.52	48.0	7.52	21.0	7.62	15.0	9.83	12.0	10.00	8.2
Aug. 21	1.22	41.4	7.87	14.2	9.21	13.4	9.98	10.2	9.36	9.6
Sept 4	1.25	22.6	6.87	8.3	8.19	6.5	10.57	7.6	9.20	5.8
Sept. 18	1.61	23.4	7.83	11.7	8.67	7.8	9.06	7.5	9.22	5.6
Oct. 2	2.02	24.8	8.04	9.0	8.55	5.9	9.15	5.5	9.30	4.2

* Parts per million
† A higher degree of removal of CO_2 has been secured by the plant in practice

Table 255. Results of Deferrization at Middleboro, Mass.
Parts per million

Determination	Well water	Subsiding basin effluent	Filter effluent
Turbidity....	10.1	7.0	1.6
Color	30.1	17.6	4.6
Oxygen consumed.	1.98	1.60	1.26
Iron	2.38	0.67	0.20
Manganese	0.71	0.31	0.12
Carbon dioxide	45.9	5.6	6.4
Dissolved oxygen	1.20	10.14	9.85

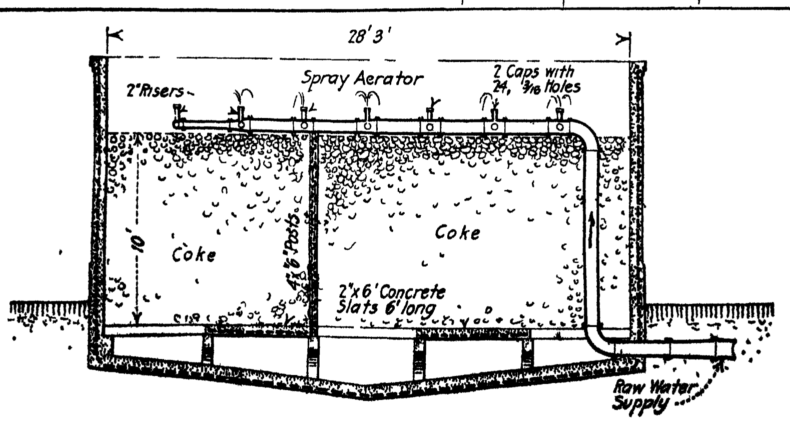

FIG. 279.

Figure 279 shows a typical trickler for use in connection with a deferrization plant. The quick-opening valve on the drain is not shown. The coke is supported on concrete slats, and water is distributed over the surface of the coke by sprays. The diameter of the coke varies from 1 to 2 in.

Demanganization.* Chemically, manganese is closely related to iron; it reacts and precipitates more slowly and possesses the power of preventing the removal of the last traces of iron unless itself be removed at the same time. The process is similar to the deferrization process, but more thorough preliminary treatment is necessary.

Table 256. Data Relating to Typical Deferrization Plants

Name of plant	Urbana, Ill.	Middleboro, Mass.	Iowa City, Iowa
Plant put in operation	1913	1913	1910
Designer	A. N. Talbot	R. S. Weston	N. Y. Continental Jewell Filtration Co.
Population (1910)	20,666 (including Champaign)	8,214	10,091
Total cost	———	$18,000	$26,000
Cost per million gals. capacity.	———	$18,000	$13,000
Source of supply	Wells	Well (near Nemasket river)	Infiltration galleries in bed of Iowa river
Rated capacity, gals. per day.	2,000,000	1,000,000	2,000,000
Total capacity of subsiding or coagulating basins, gals.	250,000*	40,000	250,000
Total capacity of filtered water basin, gals.	700,000*	42,000	
Chemicals used	Aeration only, no chemicals	Aeration only, no chemicals	Lime
Manner of application of chemicals.	....	....	applied near entrance to settling basins
Number of filter units	4	2†	4
Net area of filter surface, sq. ft.	720	4,350	700
Depth of filtering materials, in.	Gravel (8), Sand (30)	Gravel (12), Sand (36)	Gravel (12), Sand (28)
Size of filtering materials, mm.	———	Eff. size 0.31, uniformity coef. 1.80	Eff. size 0.54, uniformity coef. 1.4
Method of cleaning	Reverse flow of water with agitation by compressed air. Occasionally all of the sand is taken out and washed	Scraped by hand	Reverse flow of water with agitation by compressed air. Vertical velocity of wash-water 1 ft. per minute
Number of subsiding or coagulating basins	1	1	2
Control of rate of filtration.	Controlled by orifice on raw-water pipe, head being regulated by hand-operated valve	Orifice type controlled by hand-operated valve	Closed type of controller

* These basins were in existence when iron removal plant was built and are used in connection therewith. † Also aerator and trickler consisting of bed of coke 30 ft. diam. and 10 ft. deep, see p. 649.

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* See Weston, *J. N. E. W. W. A.*, March, 1914.

CHAPTER 33

FILTRATION

A filter consists of a layer of filtering material, generally sand, through which water is passed for the purpose of removing bacteria and other suspended matter. Said layer is supported by an underdrain system within a vessel or basin provided with various accessories. Filters act primarily as strainers, but the finest particles of silt and the largest bacteria are greatly inferior in size to the smallest sand grains (see Fig. 280). Removal of fine suspended matter, including bacteria, is accomplished in part by the absorptive power of the surface film which surrounds each sand grain, particularly the sand grains in the upper layers of the bed. Another force which assists in the removal of suspended matter is the film which condenses on each particle of suspended matter. These films tend to coalesce with those surrounding the

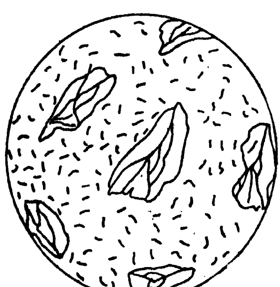


FIG. 280.—Largest bacteria (Megatherium) and smallest sand particles drawn to same scale. (×50.)

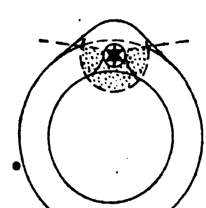


FIG. 281.—Coalescence of films on large and small particles. (Pennink.)

sand grains (see Fig. 281). Another important factor is the orderly arrangement or orientation of water molecules against all surfaces, forming a thin stagnant layer which resists the passage of solids. It was formerly thought that the surface film layer, or "schmutzedecke," accomplished the removal of all the suspended matter, but experience has shown that thicker beds are steadier in operation; it is now believed that each particle of sand with its accumulated film, plays a more or less important rôle in the removal of suspended matter, although the greater part of the suspended matter is always removed in the surface layer of sand.

The two principal classes of filters are the slow ("slow sand" or "English" or "sand"), and the rapid (or "rapid sand" or "mechanical") filters. Slow filters differ from rapid filters in that the rate of filtration is much lower and the filter is cleaned by scraping off the superficial layer, which is washed and replaced either immediately or at some convenient time, while rapid filters are cleaned by reversed flow at high velocity. Well-designed and well-operated

filter plants can be depended upon to produce an agreeable and safe water under practically all conditions.

Water Purification Statistics. Pub. Works, Vol. 54: 1923, pp. 399–401. Data are tabulated from 70 cities. Ninety-three per cent. use Cl; 56 per cent. filter. See also "Filtration Plant Census," 1924, in J. A. W. W. A., Vol. 14, 1925, p. 123.

SAND* AND GRAVEL

Filter sand† should be hard and resistant, preferably of quartz or quartzite, and free from excessive quantities of fine particles and dirt of every description. Filter sand should not contain more than 2 per cent. of lime and magnesia calculated as carbonates. It usually requires washing and screening before placing in the filter.

Size of sand grains‡ is determined by sifting through a set of rated sieves. About 110 g. of moist sand are put in a small iron dish and dried in an oven. After cooling, 100 g. are put in the coarsest of a set of sieves, and the sieves are put in a mechanical shaker. A definite number of turns found by experience to be sufficient, is given. Shaking is not continued until no more passes, but only until the amount passing is small, so that doubling the number of shakes would not greatly change the result. The sieves are then taken apart, the material that has passed all the sieves is first put upon the pan of the scale and weighed, then the material remaining on the finest sieve is added to it and again weighed. The process is continued until all the material is on the scale, when it should equal the original weight. The percentages finer than the sizes corresponding to the several sieves are then plotted on a diagram, from which the required data are taken.

Effective size of sand is the size which is coarser than 10 per cent of the sand grains by weight. The size of a sand grain is always taken as the diameter of a sphere of equal volume.

Uniformity coefficient is the ratio between the size such that 60 per cent of the sand is finer than it and the effective size. (See also pp 709 and 720)

In rating sieves, an ordinary sand is put upon them and the shaking is performed with the usual number of revolutions. The sieves are then taken apart, each sieve is taken separately, and is given a further slight shaking. A small additional amount of sand passes. The grains so passing are substantially larger than all the grains that have previously passed and smaller than those that remain. This small quantity of sand represents the size of separation of the sieve. A certain number of sand grains are counted out and weighed on a fine balance, and the average weight obtained. The diameter of grain of average size is obtained by the formula

$$D \text{ (in mm.)} = 0.9\sqrt[3]{w} = \sqrt[3]{\frac{6}{\text{sp. gr.} \times \pi}}\sqrt[3]{w}$$

w = weight in milligrams.

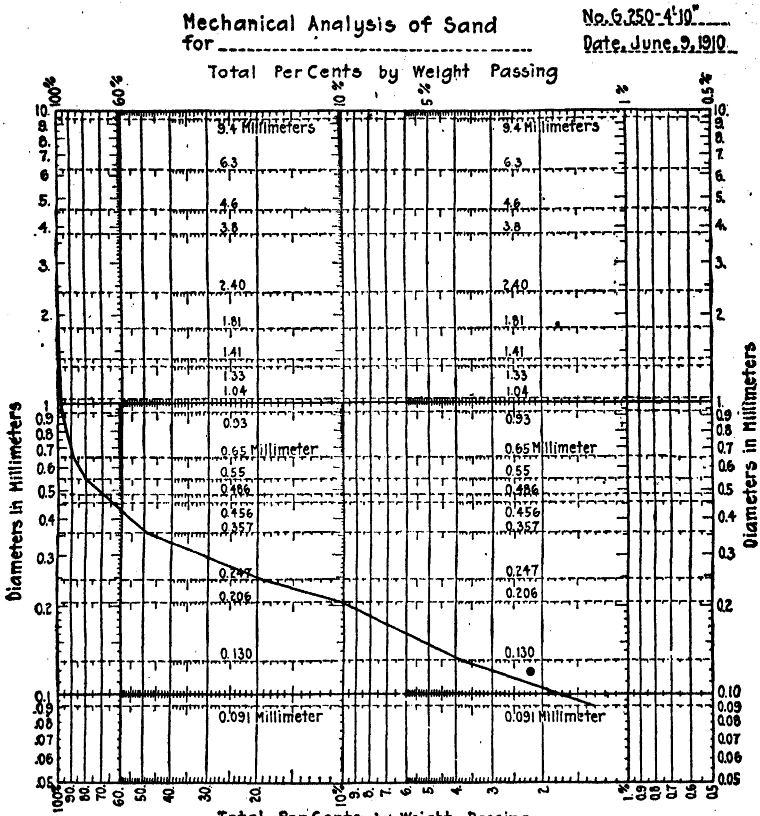
There is a little difference in the results of using round-grained and sharp-grained sands, and between grains of different shapes, but the rating is best carried out with various representative sands. Rating of sieves once made does not change appreciably with use until some openings become enlarged or some wires become broken. When this happens the sieves should be at once replaced.[8]

* See also Report of Investigations, 2622 (1924), U. S. Bureau of Mines

† See *Report* of Committee, *J A W W A*, Vol 11, 1924, p 677

‡ From Hazen, "American Civil Engineers Hand Book," Mansfield Merriman, editor (John Wiley and Sons, Inc., 1920), pp 1214–1215.

Sand Analysis. Figure 282 illustrates the method of analyzing sand and the results obtained thereby; it also shows, approximately, the relation between certain commercial brass wire cloths and punched metal and the sizes of sand separated thereby. Standard cloths should be used if possible.



Mechanical Analysis

Date of Collection, June 9, 1910

Date of Analysis, June 15, 1910

	Size of Size: Sieve	Size of Size: M.M.	Amt. Passing: Grams	Amt. Passing: Per Cent
Woven Brass Wire	200.	0.091	2.6	1.3
	140.	0.130	7.5	3.8
	100.	0.206	20.0	10.3
	80.	0.247	38.3	19.6
	60.	0.357	94.0	48.2
	Cu. 40.	0.456		
	40.	0.486	136.5	70.0
	35.	0.55	157.1	80.6
	30	0.65	174.5	89.5
	Cu. 26	0.66		
	25.	0.93	188.4	96.7
	20.	1.04	191.5	98.3
	16.	1.41	193.7	99.2
Punched Metal	0.057"	1.33		
	0.079"	1.81		
	0.103"	2.40	194.3	99.7
	0.156"	3.8		
	0.187"	4.6		
	0.25"	6.3		
	0.375"	9.4		
Total			194.3	99.7
Loss			0.7	0.3
Amount Sieved			195.0	100.0

60% Finer than 0.42 mm.

10% Finer - Eff. Size 0.20 mm.

Uniformity Coefficient 2.1

Analyst S.A.W.

FIG. 282.—Chart for sand analysis.

Turbidity of sand is the measure of clay in it in p.p.m. Put 10 g. of moist sand into a glass vessel holding 1 liter and fill with clear water. Agitate vigorously until all fine matter is in suspension. Allow to settle 1 min., take the turbidity with a rod, and multiply the result by 100. The observed turbidity × 100 = required turbidity. The weight of clay is from ⅓ to ⅔ the turbidity, depending

upon the size of the clay particles. Turbidity of sand prepared from stock containing clay should always be taken, but when there is no clay in the stock it is unnecessary to do it. Turbidity of slow filter sand should not be allowed to exceed 4000 p.p.m., or 0.4 per cent., corresponding to about 0.2 per cent. actual clay.*[3b]

Preparation of Filter Sand and Gravel. Sand for filters can be obtained from a wide range of raw material. Occasionally sand may be found which is of such size, uniformity, and degree of cleanliness that it may be used without screening or washing. Other sands require special preparation. Sand may be dredged from river bars or beaches and pumped through screens or riddles to remove the coarser particles. The finer particles, including clay, may be most conveniently removed by washing the sand in a box deeper at one end than at the other and provided with pipes for distributing the water along its bottom. The dirty sand is dumped at the shallow inlet end and the clean sand is taken out through a sand valve at the deeper outlet end. Meanwhile, the wash water moves upward perpendicularly to the path of the sand and carries with it the dirt and finer particles. Sand containing as high as 10 per cent. of fine material may be prepared by this method.[5]

Sizes of Separation. The *hydraulic value*† of sand of sizes ordinarily used in water filters is approximately as follows: Effective size in mm. × 100 = hydraulic value in mm. per sec. The size of separation in sand-washing boxes depends upon the area of the box and the method of distributing the wash water. The approximate size of separation such that 75 per cent. of the particles of that size will be retained, may be computed by the formula

$$D \text{ in mm.} = 0.065 f \frac{\text{g.p.m. of water overflowing}}{\text{sq. ft. of box area}}$$

f = factor determined by size and shape of box, and varies from 3.0 for an ordinary box to 1.5 for a well-designed box with uniform distribution of wash water.‡[3a]

Voids in filter sand vary from 35 to 45 per cent., according to uniformity coefficient and method of placing. Loosely placed sand may settle as much as 15 per cent. after a filter is filled from below with water. Moist, packed sand settles from 4 to 8 per cent. when thus filled with water. Sand placed by hydraulic methods or when perfectly dry, packs closely. *Determination of voids* in sand is made by driving a sheet iron cylinder into the sand so as to fill it completely; the sand removed from the filled cylinder is dried, weighed, and the volume of the solid particles computed (sp. gr. sand = 2.65). The volume of sand is compared with the volume of the cylinder.

SLOW FILTERS

Structural Features.§ Slow filters are masonry basins, usually covered, varying in area from a few square feet to 1.5 acres, filled with sand, underdrained, and provided with pipe connections and appurtenances. The main

* Discharge coefficient (c) for any gravel is 1000 Q (Q = mgal passing when $\frac{\text{head lost}}{\text{distance traveled}}$ = 0.001). Friction head in gravel = $\frac{1}{2} \frac{Q \times r^2}{\text{Average gravel depth} \times c}$. (See Fig 228.) r = ½ distance between drains.[4]

† Velocity required to float the particle, in mm per second.

‡ For sand for rapid filters, see pp 720.

§ See also p. 538.

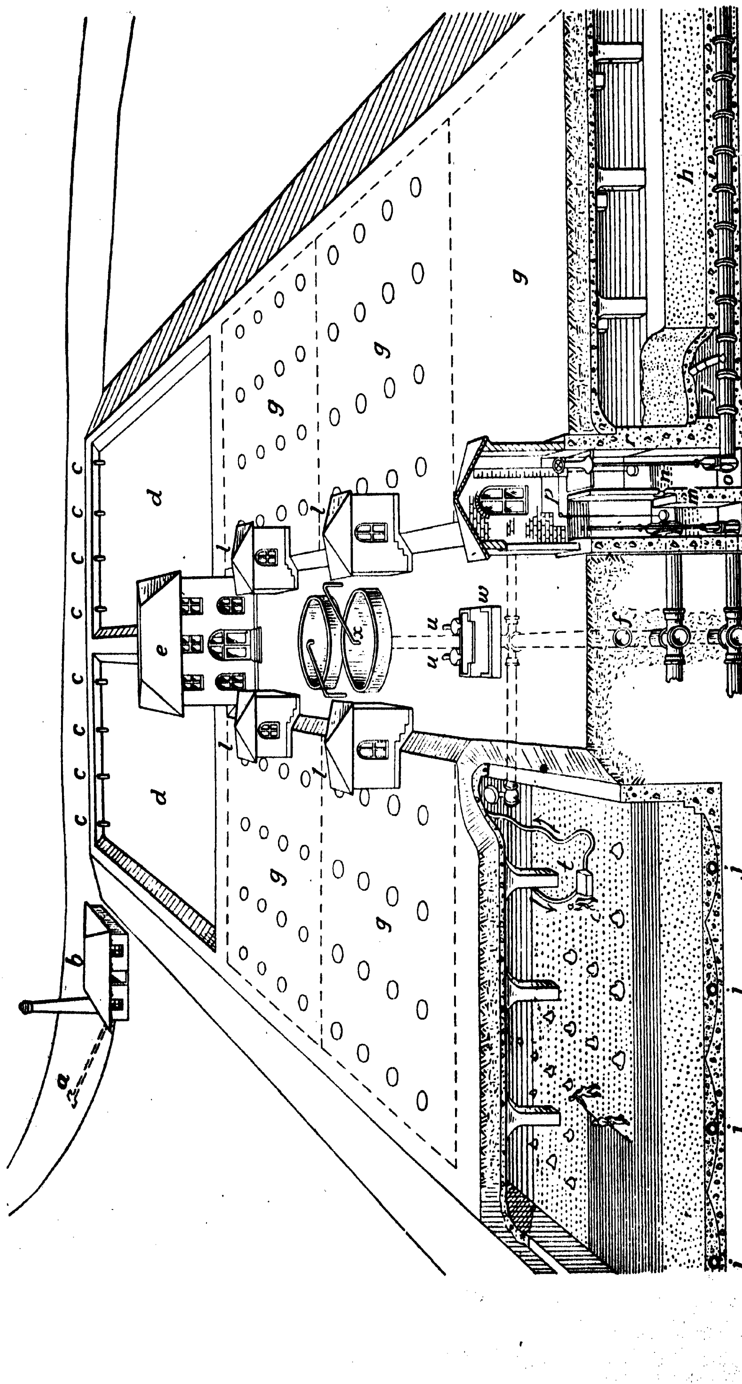


FIG. 283.—General view of slow filter plant.
(Reproduced by permission from "Water Purification Plants and Their Operation," by Milton F. Stein, p. 23, published by John Wiley & Sons, Inc., 1926.)

drains are usually built into the masonry bottom, while the connecting laterals are laid on its surface and are covered to a depth of a foot or more with three or more layers of gravel to support from 2 to 5 ft. of filter sand. Figure 283 shows a typical plant. It consists of duplicate sedimentation basins, *d*, the filter units, *g*, the laboratory *e*, etc. Low-lift pumps, *b*, take water from intake, *a*, and deliver through aerating risers, *c*, to sedimentation basins, *d*. Water is uniformly distributed to filters, *g*, by a float valve. Collector pipes, *j*, collect the filtered water and lead to the main collector, *K*, which leads to the regu-

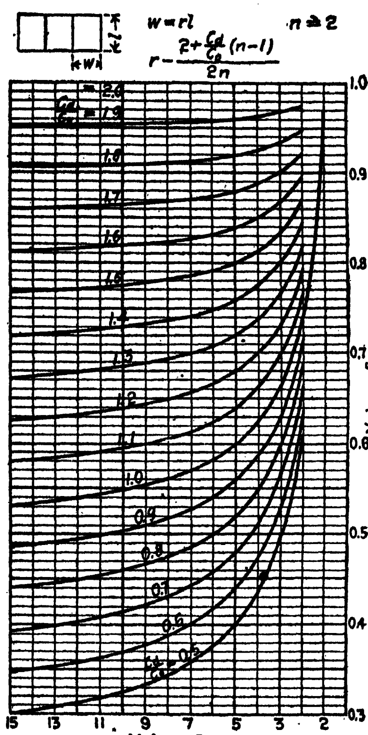


Fig. 284.—Single series of beds.

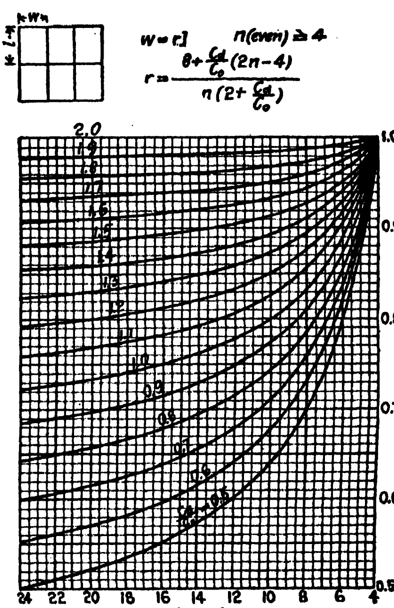


Fig. 285.—Double series of beds.

lator house, *l*, whence it flows through regulating orifices to the clear water well. A portable sand ejector, *t*, is employed when the sand requires washing. Sand washers, *u*, discharge the clean sand to sand storage bins, *x*.

Principle. Water usually enters the filter through a float valve or other device for maintaining a depth of from 2 to 5 ft. above the sand surface. The effluent discharges through a gate house in which are located various gates and valves, and devices for regulating the flow according to loss of head and rate of filtration. The passage of the water through the sand is attended with loss of head which gradually increases as the filter becomes clogged. When clogging has reduced the working head to the practical minimum, the influent valve is shut and the filter is allowed to drain until the sand surface is exposed, when the surface of the filter is raked to facilitate passage of water, or is

cleaned by scraping off a thin layer of sand which is usually washed and replaced.

Slow vs. Rapid Filtration. The chief *advantages* of slow filtration over rapid filtration are: (1) The application of chemicals is generally obviated; (2) The effluent is usually less corrosive; (3) the mechanisms are less intricate; (4) less and less expert supervision is required; (5) where no disinfectants are used, the bacterial efficiency is higher. The chief *disadvantages* are: (1) the large area required; (2) the higher cost especially in northern climates, where a roof is needed; (3) low efficiency of color removal (20 to 50 per cent.); (4) less flexibility in meeting emergency demands; (5) poor results if turbidity exceeds 40 p p.m.

Economical Dimensions of Rectangular Filter Beds. Gregory[1,2] gives diagrams, Figs. 284 and 285, p. 707, computed from the following data: n = number of beds; A = total area of all beds in sq. ft, w = width of 1 bed in ft.; l = length of 1 bed in ft.; r = ratio of width to length of 1 bed; C_o = cost of outside walls per lin. ft.; C_d = cost of dividing walls per lin. ft.; C = total cost of walls. Figure 284 applies to a single series, Fig 285 to a double series of beds. *Number and size of beds* in small filter plants are matters requiring judgment more than computation.

Area of individual filters depends upon the capacity of the whole plant, and should be small enough so that at least one filter in a small, or two in a large, plant can be out of service at all times for cleaning.

Underdrain systems must be so designed that, at the nominal rate of filtration, the frictional resistance of the whole system will be about 25 per cent

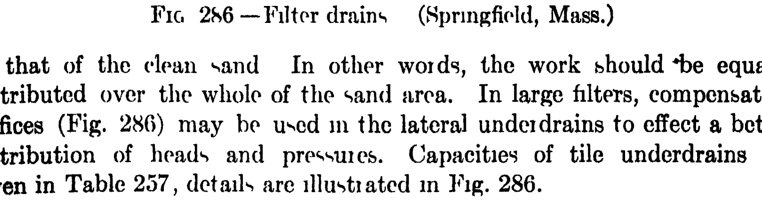


FIG 286 —Filter drains (Springfield, Mass.)

of that of the clean sand In other words, the work should be equally distributed over the whole of the sand area. In large filters, compensating orifices (Fig. 286) may be used in the lateral underdrains to effect a better distribution of heads and pressures. Capacities of tile underdrains are given in Table 257, details are illustrated in Fig. 286.

Table 257. Underdrains for Slow Filters (No Compensating* Orifices Used)

Rate of filtration, million gallons per acre daily	5	6	8	10	15
Average resistance of clean sand in feet	0 150	0 180	0 240	0 300	0.450
Total allowable friction and velocity head in underdrainage system, feet	0 037	0 045	0 060	0.075	0.112
Approximate ratio of filter area to area of main drain	5100	4700	4200	3800	3200
Approximate maximum velocity in main drain (varying somewhat with size), ft. per sec	0 90	1 00	1 18	1.34	1.68
Approximate maximum velocity in laterals (varying somewhat with size), ft. per sec	0.55	0.61	0.72	0.82	1.04

* See Fig. 286.

Table 258. Maximum Areas of Filter Beds Drained in Square Feet[a]

Diameter of drain, inches	Shape and kind of drain	Rate of filtration, million gallons per acre per day				
		5	6	8	10	15
4	Round lateral	264	245	218	200	168
5	Round lateral	420	390	345	316	266
6	Round lateral	610	570	500	460	390
8	Split lateral	520	490	430	400	320
10	Split lateral	830	770	680	630	530
12	Split lateral	1,200	1,120	1,000	910	770
10	Round main	2,700	2,500	2,200	2,000	1,700
12	Round main	3,900	3,600	3,200	2,900	2,400
15	Round main	6,200	5,800	5,100	4,600	3,900
18	Round main	9,000	8,300	7,400	6,700	5,600
21	Round main	12,300	11,400	10,000	9,100	7,600
24	Round main	16,100	14,900	13,200	12,000	10,000
36	Round main	37,000	34,000	30,000	27,000	22,000

Masonry roofs with earth covers are required for all cold regions, generally speaking, for all cities north of the Potomac and Ohio Rivers. South of this line, filters may have to be covered with light-tight roofs to prevent growths of algae. Typical cross-sections of covered filters at Middletown and Springfield, Mass., and Washington, D. C., are shown in Fig. 287, which also shows cross-sections of two, covered, filtered-water reservoirs. The groined arch is usually the cheapest form of roof which affords sufficient head room above the sand, although in some places* a reinforced slab roof on reinforced-concrete columns, as used for reservoirs, was considered advisable (see discussion on p. 547).

Gravel is placed usually in three or four layers, lowest layer about 7 in. thick of a size passing a 2-in. and retained on a $\frac{3}{4}$-in. screen, effective size about 20 mm.; middle layer 3 in. thick of a size passing a $\frac{3}{4}$-in and retained on a $\frac{3}{8}$-in. screen, effective size about 8 mm.; and top layer 2 in thick, of a size passing a $\frac{3}{8}$-in. screen, effective size 2 to 3 mm. The effective size of the supported layer should be at least $\frac{1}{4}$ of that of the supporting layer.

Gravel should be omitted about the piers and along the walls for a width of 2 ft. to lessen the chance of unfiltered water reaching underdrains without first passing through sand. For the same reason piers and walls should be battered or stepped.

Gravel can be obtained from a wide range of raw material. It should be prepared by screening and washing. Crushed rock may be used where gravel is not obtainable. Often enough gravel may be sieved from the source of filter sand. Figure 288 shows the values of the discharge coefficient (c) for gravel of different effective sizes (p. 705).

Sand. Ordinarily the sand for slow filters should have an effective size of from 0.25 to 0.35 mm. and a uniformity coefficient not over 3.0. It should contain not much more than 2 per cent. of calcium and magnesium computed as carbonates.

Washing and Handling Filter Sand. In all types of filters, sand is usually handled and washed hydraulically. Sand scraped from slow filters is usually

* Montreal, Keene, N H, New Haven, Conn

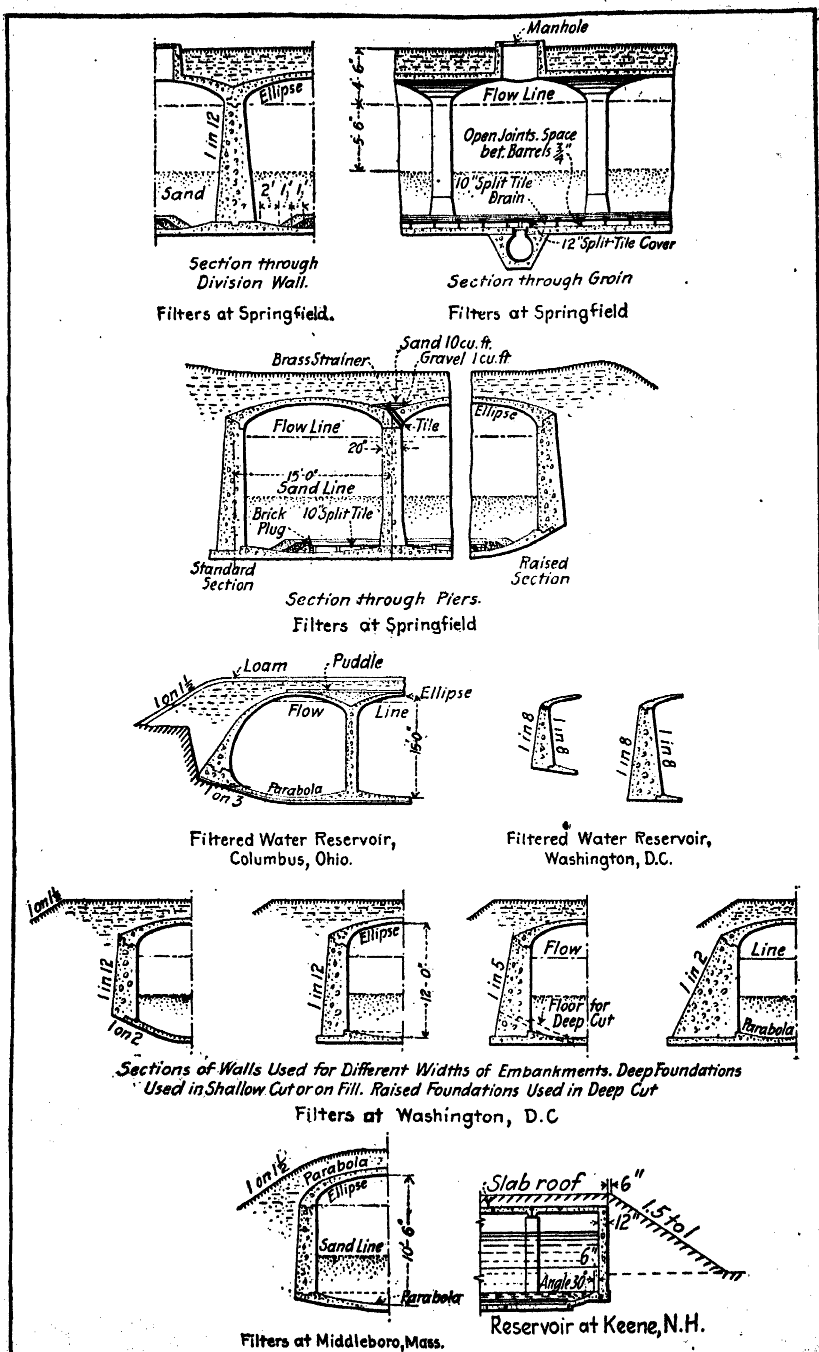


FIG. 287.—Masonry structures of typical reservoirs.

gathered into piles and removed from the filter by means of portable ejectors, Fig. 289. Each consists of a tight metal box carrying a large ejector operating with water under pressure furnished through a short 2½-in. hose attached to one of several hose connections in a 3-in. or 4-in. high-pressure main running through the filter. The sand is shoveled into the ejector, usually through a screen, to prevent the entrance of gravel into the throat of the ejector. Sand is kept in a suspended state by perforated irrigating pipes in the bottom of the box. It is kept from packing and arching by sprays impinging upon the top sand in the ejector. A mixture of sand and water passing through the throat of the ejector is conveyed through discharge hose and connecting piping

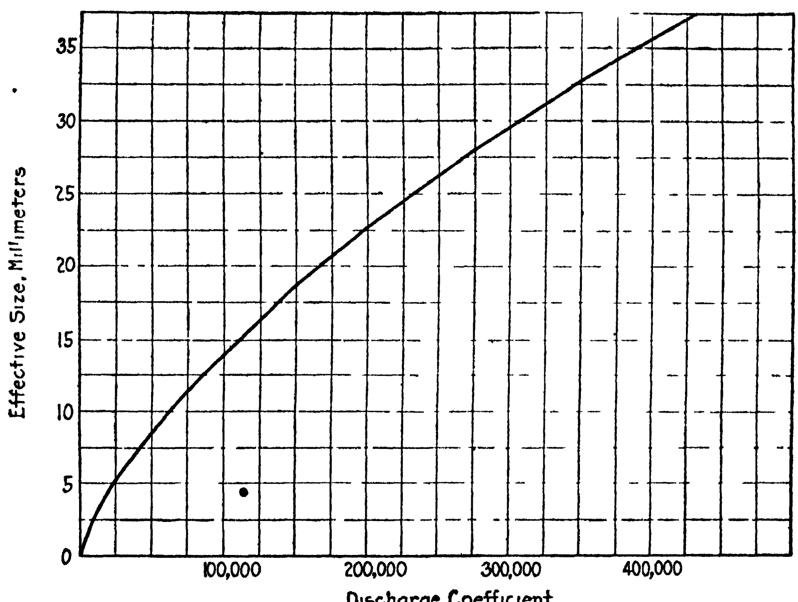


FIG. 288 Values of c for gravel.

to a sand-washing machine consisting of two or more hoppers. In these hoppers, dirt and fine particles of silt are washed from the sand and then the clean sand is discharged by the ejector at the bottom of the final hopper, to a storage pile in a sand court, to elevated storage bins, through a special device for separating sand from water which can be placed over a filter manhole and discharge the sand directly into the filter, or directly into another filter which is out of service. In the latter case, the sand may be discharged through multiple nozzles at the end of the sand hose supported by a boat or by a carriage supported on tracks within the filter. Meanwhile, water is passed slowly through the filter from below, escaping over the wall of the inlet chamber. The water surface serves as a guide for leveling the sand.

Per cent. of water for cleaning sand, etc., varies from 0.25 to 1 per cent. of the water filtered.

The resistance to be overcome in the sand discharge piping is made up of actual lift and friction. For the lift, multiply the actual lift in feet by the specific gravity of the mixture. For friction of sand and water in 3- and 4-in. pipes, compute the friction for water alone, and add 3.5 ft. per thousand for each per cent. of sand in the mixture. For 6-in. or larger pipe add 2.5 ft. and for 2.5-in. hose, add 4.5 ft. per thousand. These figures will be close enough for velocities 5 ft. per sec. or over. Sand and water mixtures will flow well at all velocities above 5 ft. per sec. and fairly well from 4 to 5 ft. per sec. Between 3 and 4 ft. per sec. there will be more friction than calculated and some stoppages; and below 3 ft. per sec. filter sand and water mixtures will not flow.

$$\left.\begin{array}{l}\text{Velocity in}\\ \text{ft. per sec.}\end{array}\right\} = \frac{137.5 \text{ cu. yd per hr}}{\text{Per cent of sand in discharge} \times (\text{diam of pipe in in.})^2}$$

$$\text{Cu. yd. per hr. 3d} = \frac{\text{Per cent. sand} \times \text{velocity} \times \text{diam}^2}{137.5}$$

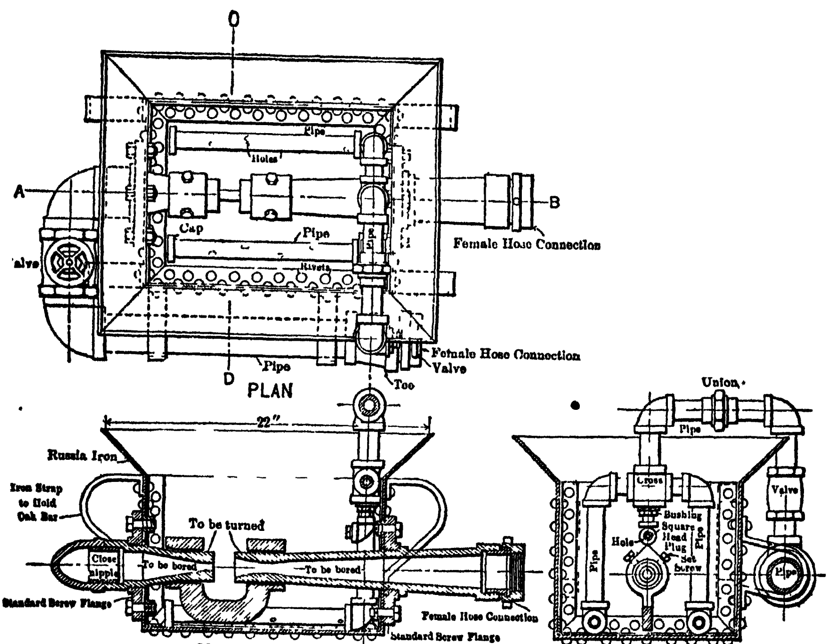


FIG. 289.—Portable sand ejector.
(Stein's "Water Purification")

Hydraulics of Sand Handling. Theoretically the best form for the throat of an ejector is that which approximates the Venturi tube. Given this shape and the best size for any given addition, the approximate relations are those given in Table 259.

Ejector throats wear rapidly, and the efficiencies with somewhat worn throats would be those computed by the formula below (Table 260). Sometimes throats are lined with soft rubber to lessen wear, or readily replaceable wrought-iron nipples are used.

Table 259. Sand Ejectors and Flow of Water and Sand in Discharge Piping[3c]

Lbs. pressure feed water	Diam. jet, in.	Best diam. for throat, in.	Per cent. sand in discharge by volume	Cu. yds. sand per hr.	Pressure of discharge in ft.	Friction in ft. per 1000 in discharge piping				Lbs. pressure feed water	Diam. jet, in.	Best diam. for throat, in.	Per cent. sand in discharge by volume	Cu. yds. sand per hr.	Pressure of discharge in ft.	Friction in ft. per 1000 in discharge piping			
						2½ in.	3 in.	4 in.	5 in.							2½ in.	3 in.	4 in.	5 in.
60	0.5	0.87	20	5.0	28	150	140	.....	...	100	0.5	0.87	20	6.4	47	162	126	.....	...
	0.5	1.01	25	7.2	20	176	150	.....	...		0.5	1.01	25	9.2	34	200	143	.....	...
	0.5	1.21	30	10.0	14	206	168	.....	...		0.5	1.21	30	12.9	23	250	164	120	...
	0.6	1.04	20	7.2	28	178	124	90	...		0.6	1.04	20	9.2	47	230	133	87	...
	0.6	1.21	25	10.3	20	222	144	110	...		0.6	1.21	25	13.3	34	300	165	103	...
	0.6	1.46	30	14.3	14	275	170	120	...		0.6	1.46	30	18.5	23	380	210	118	105
	0.7	1.06	15	6.5	39	200	114	70	...		0.7	1.06	15	8.4	65	292	144	71	...
	0.7	1.21	20	9.7	28	250	140	88	..		0.7	1.21	20	12.6	47	360	180	88	71
	0.7	1.41	25	14.0	20	325	175	102	...		0.7	1.41	25	18.1	34	470	235	108	83
	0.8	1.21	15	8.5	39	298	145	71	..		0.8	1.21	15	11.0	65	450	208	82	56
	0.8	1.39	20	12.7	28	370	180	89	73		0.8	1.39	20	16.4	47	550	260	104	71
	0.8	1.61	25	18.4	20	480	240	109	82		0.8	1.61	25	23.7	34	.....	340	130	86
	0.9	1.20	10	6.5	52	350	160	61	42		0.9	1.20	10	8.3	87	.....	240	78	45
	0.9	1.37	15	10.8	39	435	202	80	56		0.9	1.36	15	13.9	65	.....	300	100	60
	0.9	1.56	20	16.1	28	540	250	102	71		0.9	1.56	20	20.8	47	.....	370	128	76
80	0.5	0.87	20	5.7	38	154	130	.....	...	150	0.5	0.87	20	7.8	71	195	124	90	...
	0.5	1.01	25	8.3	27	186	145	.....	...		0.5	1.01	25	11.4	51	240	150	110	...
	0.5	1.21	30	11.5	18	225	160	..	...		0.5	1.21	30	15.8	35	305	180	118	...
	0.6	1.04	20	8.3	38	208	128	90	..		0.6	1.04	20	11.3	71	310	160	86	75
	0.6	1.21	25	11.9	27	260	153	107	...		0.6	1.21	25	16.3	51	400	205	104	83
	0.6	1.46	30	16.5	18	320	187	117	..		0.6	1.46	30	22.7	35	520	260	125	100
	0.7	1.06	15	7.5	52	245	127	70	..		0.7	1.06	15	10.3	98	410	190	78	56
	0.7	1.21	20	11.2	38	305	158	86	...		0.7	1.21	20	15.4	71	500	236	98	71
	0.7	1.41	25	16.2	27	400	204	104	71		0.7	1.41	25	22.2	51	640	310	120	85
	0.8	1.21	15	9.8	52	370	176	76	56		0.8	1.21	15	13.5	98	.....	285	99	59
	0.8	1.39	20	14.7	38	465	222	95	71		0.8	1.39	20	20.0	71	.....	350	123	75
	0.8	1.61	25	21.2	27	600	290	115	84		0.8	1.61	25	29.0	51	.....	450	160	93
	0.9	1.20	10	7.5	70	450	200	70	43		0.9	1.20	10	10.2	131	.....	335	100	50
	0.9	1.36	15	12.4	52	550	248	91	58		0.9	1.36	15	17.0	98	.....	410	129	67
	0.9	1.56	20	18.6	38	680	315	114	73		0.9	1.56	20	25.5	71	.....	510	163	85

Hazen, "Am. C. E. Handbook," Mansfield Merriman, editor, John Wiley and Sons, Inc., 1920, pp. 1217–1218.

Table 260. Sand Handling Data[36]

Per cent. of sand in water thrown by vol.	5 0	10 0	15 0	20.0	25.0	30.0
Specific gravity of mixture..............	1.05	1.10	1.15	1.20	1.25	1.30
Per cent. slush by volume...............	8.3	16.7	25.0	33.3	41.7	50.0
Per cent. nozzle water by volume........	91.7	83.3	75.0	66.7	58.3	50.0
Weight of slush† per part water from nozzle	0.15	0.32	0.53	0.80	1.14	1.59
Q = ratio total weight of discharge to weight of jet water..................	1.15	1.32	1.53	1.80	2.14	2.59
P = proportion of jet pressure developed in discharge	0.50	0.37	0.28	0.20	0.14	0.10
T = ratio of diameter of throat to diameter of jet............................	1.18	1.33	1.51	1.73	2.02	2.43
V = ratio of velocity in throat to velocity in jet	0 79	0.68	0.59	0.50	0.42	0.35

The formula $(P + V)T^{1.5} = 1.65$ applies to well-shaped Venturi throat ejectors throwing sand The best results are obtained when $QV = 0.9$ with 5 or 10 per cent variation either way, and within this approximate range $PQ^2 = 0.65$ This is the most convenient equation for comparing efficiencies

† Slush = suspension of sand in water

Table 261. Sizes and Capacities of Nichols Sand Separators

Size	Maximum capacity per hour	Height	Extreme diameter of frame	Weight	Pipe connection
30 in.	7 cu. yds.	5 ft. 0 in.	3 ft. 8 in.	500 lbs.	3 in.
36 in.	10 cu. yds.	5 ft. 6 in.	4 ft. 0 in.	600 lbs.	3 in.
42 in.	15 cu. yds.	6 ft. 4 in.	4 ft. 6 in	700 lbs.	3 in.

Nichols sand separator (Fig. 290), is a device which combines the functions of both hopper washer and sand separator. Sand and water are conveyed to

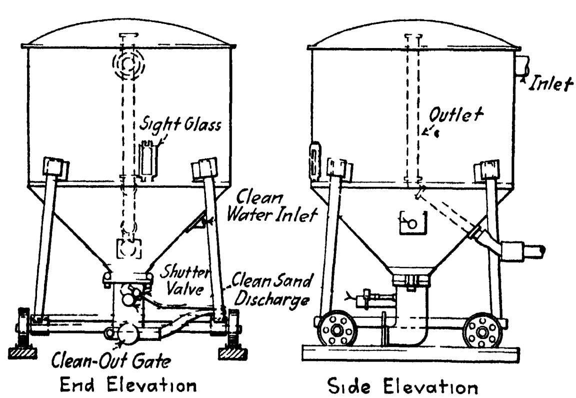


FIG. 290.—Nichols sand separator.

this machine by means of an ejector (see p. 712). As the mixture, containing from 10 to 25 per cent. of sand, passes into the separator, it strikes a series of baffles which precipitate the clean sand to the bottom, where it may be discharged through a valve, while the dirty water discharges through a pipe at the top. The sand is returned to another part of the filter; any depth may be washed.

Blaisdell washing machine cleans the sand in a slow filter by methods analogous to those used for washing mechanical filters. In its newest form, the Belt-Tread Filter Washer, it consists of a gasoline-engine-driven tractor, traveling over the sand surface and carrying a washing box, 6 by 1 ft., contain-

ing six washing units, each consisting of a system of eight revolving teeth and jets. Wash water from a hose is forced through the revolving jets and is sucked away from the washing box by a pump which connects through a hose with the sewer. As the washer is driven over the filter, with 8 in. of water above the sand surface, it washes successive areas of sand to a depth of 8 ft. With its use, rates of filtration as high as 30 mgad. may be practicable, while the consumption of wash water may be less than 2 per cent. Installations with the washing chamber suspended from a traveling crane are at Montreal, Que., Wilmington, Del., and Calexico, Cal. The machines in use require considerable repairing for efficient operation.

The cycle or period of a slow filter is the time between cleanings expressed in millions of gallons per acre. That is to say, if a filter operates 20 days at a 5-million rate, the cycle is 100. The average cycle for a plant is found by dividing the number of million gallons filtered in 1 year by the total number of acres of filter surface cleaned. Cycles are increased by drawing the water off when the loss of head has reached the allowed limit, and raking the surface of the sand and then proceeding. It does not usually pay to rake more than once in one cycle. Thorough sedimentation lengthens the cycle. Application of coagulant lengthens it, if applied sufficiently long before filtration, but with a short period of coagulation after application it shortens it. Preliminary filters lengthen the cycle. Cycles commonly range from 50 to 200; if they average less than 50, there is something wrong with the arrangements. Cycles are virtually the "yields of runs."

Loss of head in slow filters is commonly limited to the depth of water above the sand, 3 to 5 ft., but losses up to 5 or 6 ft. are sometimes permitted, although

Table 262. Losses of Head in Feet for Various Rates of Filtration with Clean Sand 3 ft. Thick

$c = 700 \quad t = 50°F.$

Rate of filtration million gals. per acre daily	Effective size of sand—millimeters								
	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60
1	0.10	0.06	0.04	0.03	0.02	0.02	0.02	0.01	0.01
2	0.20	0.13	0.09	0.07	0.05	0.04	0.03	0.03	0.02
3	0.30	0.19	0.13	0.10	0.08	0.06	0.05	0.04	0.03
4	0.40	0.26	0.18	0.13	0.10	0.08	0.06	0.05	0.04
5	0.50	0.32	0.22	0.16	0.12	0.10	0.08	0.07	0.06
6	0.60	0.38	0.27	0.20	0.15	0.12	0.10	0.08	0.07
7	0.70	0.45	0.31	0.23	0.18	0.14	0.11	0.09	0.08
8	0.80	0.51	0.36	0.26	0.20	0.16	0.13	0.11	0.09
9	0.90	0.57	0.40	0.30	0.22	0.18	0.14	0.12	0.10
10	1.00	0.64	0.45	0.33	0.25	0.20	0.16	0.13	0.11
12	1.20	0.77	0.54	0.40	0.30	0.24	0.19	0.16	0.13
14	1.40	0.90	0.63	0.46	0.35	0.28	0.22	0.18	0.15
16	1.60	1.02	0.72	0.53	0.40	0.32	0.26	0.21	0.18
18	1.80	1.15	0.81	0.59	0.45	0.36	0.29	0.24	0.20
20	2.00	1.27	0.89	0.66	0.50	0.40	0.32	0.27	0.22
100	10.02	6.37	4.46	3.28	2.51	1.98	1.61	1.33	1.11
125	12.52	7.96	5.57	4.10	3.13	2.48	2.01	1.67	1.39
150	15.03	9.55	6.69	4.92	3.76	2.97	2.41	1.99	1.67
175	17.54	11.14	7.80	5.74	4.38	3.47	2.81	2.33	1.95
200	20.05	12.74	8.92	6.55	5.01	3.96	3.21	2.65	2.23

this is liable to result in "air binding" due to the liberation of dissolved air in the sand. Initial losses of head with clean sand are given in Fig. 298, p. 727.

Rates of filtration employed with slow filters filtering river waters without preliminary chemical treatment are about 4 mgad. For lake and reservoir waters rates twice as high are used. For filtering river waters after preliminary chemical treatment and subsidence rates as high as 10 mgad. can be used. For deferrization, a rate of 10 mgad. is satisfactory. With auxiliary disinfection, or when Blaisdell Washers are used, higher rates are practical. There is a tendency toward higher rates.

Regulation. Rate of filtration in slow filters may be controlled by hand-operated gates and measuring devices, preferably of the orifice or Venturi type. These latter may be connected with either indicating or recording devices, the latter preferred. When Venturi meters are used, indicating and recording devices usually show both loss of head and rate of flow. Automatic controllers with hydraulic valves actuated by the Venturi meter mechanism or other controllers may be used, but are not necessary for rates less than 6 mgad.

Raking. In some filters, especially those used with deferrization plants, it is desirable to retain as large an accumulation on the surface of the sand as practicable in order to increase the efficiency of the process. In such filters, when the maximum loss of head is reached, the filter is drained, raked, and put into service without removing the sand. Raking between scrapings is also practiced to reduce cost of sand handling but is not successful with all waters, although sometimes a filter may be raked two or three times between scrapings.

Scraping. When the maximum loss of head is reached, from $\frac{3}{8}$ to 1 in. of sand is removed either by scraping and ejecting as described under "Sand Handling," or, in some old plants, by scraping into wheelbarrows. Sometimes, in cold weather, it is the practice to scrape and pile the sand and remove it from the filter at a more convenient season.

Starting. After raking or scraping,* the filter should be started slowly, and the rate gradually increased until the efficiency of the filter is established. Filters require a period of biological construction or film-forming in order to attain their maximum efficiencies. Old filters, therefore, are much more efficient than new.

RAPID FILTRATION†

Principle. Rapid filters, like slow filters, depend upon straining action and surface adsorption for their efficiency. With rapid filters, surface adsorption is increased and higher rates made practicable by applying chemicals to form flocculent precipitates, whose large surfaces attract and adsorb suspended matter and color. Where the preliminary flocculation of suspended matter is thoroughly accomplished, efficient clarification may be obtained at almost any practicable rate of filtration. Rapid filters are washed by reversing the current of water through the sand.

Types. Rapid filters may be gravity or pressure. Gravity filters are placed near the hydraulic grade line of the influent. Pressure filters are

* And refilling the sand from below.
† Rapid vs. slow, see p. 708.

placed well below the grade line, and choice is determined by hydraulic and topographical conditions Pressure filters (Fig 291), are closed cylinders of steel or iron, through which the water is forced by pressure Gravity filters are in open vessels (Fig 292), of wooden, steel, or concrete construction. Figure 295 shows a filter designed for washing at high velocity.

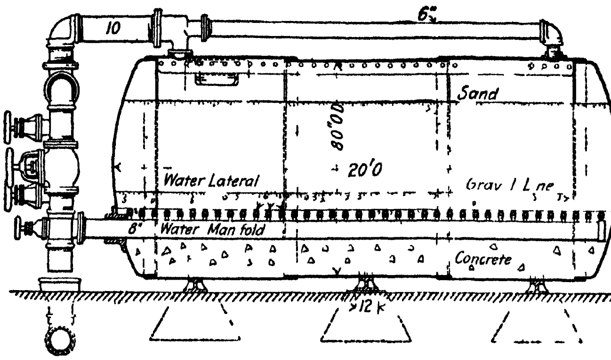
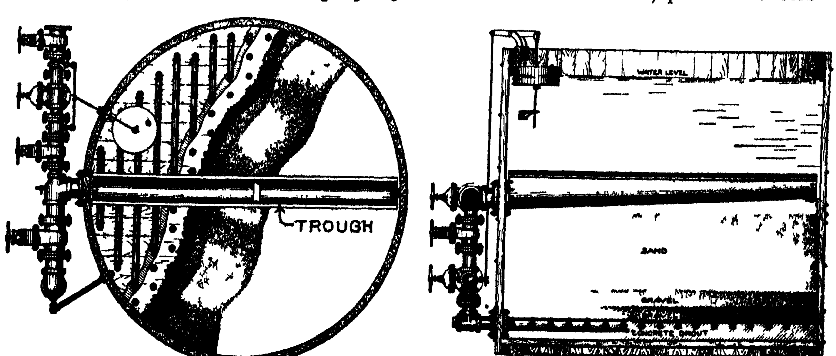


FIG 291 —Horizontal pressure filter, reverse current wash
(Am Water Softener Co)

Pressure filters* of steel or iron, through which the water is forced are generally most suitable for small installations although there are large plants at Atlanta,† East St Louis, Davenport, Iowa, Terre Haute, Ind, and in industrial plants

Pressure filters may be vertical or horizontal Where hydraulic conditions demand, and the water is properly treated before filtration, pressure filters



Plan Section
FIG 292 —Circular (wooden) gravity filter
(International Filter Co)

may be operated successfully, provided, however, the same attention be given to the design of the filter—particularly its underdrain system—and washing and regulating devices,—as is given to the design of gravity filters. Gravity filters are usually preferable, and where considerable preliminary treatment,

* See "Pressure Filters," by H Stevens *J A W W A* Vol 3, 1916, pp 388–397, 750–778; also Ellms, 'Water Purification McGraw-Hill Book Company, Inc, 1916

† Superseded by gravity filters in 1924

Table 263. Dimensions and Capacities for Steel and Cast-Iron Pressure Filters*
Vertical Filters

Size	Area,† sq. ft.	Capacities, g.p m. At gals. per sq ft. per min. 2	3	4	Pipe size, inches Inlet, outlet, wash	Waste to sewer	Minimum gallons of wash water at 12 gals. per sq ft per min
Diam. in	**VERTICAL FILTERS**						
12	785	1 57	2 35	3 04	$\frac{3}{4}$	1	9 42
14	1 06	2 12	3 18	4 24	1	$1\frac{1}{4}$	12 72
16	1 39	2 78	4 17	5 56	1	$1\frac{1}{2}$	16 68
20	2 18	4 36	6 54	8 72	$1\frac{1}{2}$	$1\frac{1}{2}$	26 16
24	3 14	6 28	9 42	12 5	$1\frac{1}{2}$	2	37
30	4 90	9 8	14 7	19 6	$1\frac{1}{2}$	2	60
36	7 06	14 1	21 1	28 2	2	$2\frac{1}{2}$	84
42	9 62	19 2	28 8	38 5	2	$2\frac{1}{2}$	115
48	12 56	25 1	37 6	50 2	$2\frac{1}{2}$	3	150
54	15 90	31 8	47 7	63 6	$2\frac{1}{2}$	3	190
60	19 63	39 2	58 8	78 5	3	4	235
72	28 27	56 5	84 8	113 1	4	5	339
84	38 48	76 9	115 4	153 9	4	5	460
96	50 27	100 5	150 8	201 1	5	6	600
Length, ft ‡	**HORIZONTAL FILTERS, 8 ft diam.**						
16	68 5	137	205 5	274 0	6	8	822
12	83 4	166 8	250 2	333 6	6	8	1000
14	98 2	196 4	294 6	392 8	6	8	1178
16	113 1	226 2	339 3	452 4	8	10	1357
20	142 7	285 4	428 1	570 8	8	10	1712
25	179 8	359 6	539 4	719 2	8	10	2157

* Recommended by the Associated Manufacturers of Water Purifying Equipment, July 7, 1922
† Area of segments of the 2 dished heads = 9 2 sq ft Area per lineal foot of bed in the cylinder = 7 42 sq ft Example 8 ft by 16 ft filter area per head = 9 2 sq ft Area in cylinder 14 by 7 43 = 103 9 Total effective area = 113 1 sq ft
‡ L = Overall length of filter and area of bed calculated for surface of bed 18 in above center of shell

Table 264. Dimensions of Vertical Steel Filters*

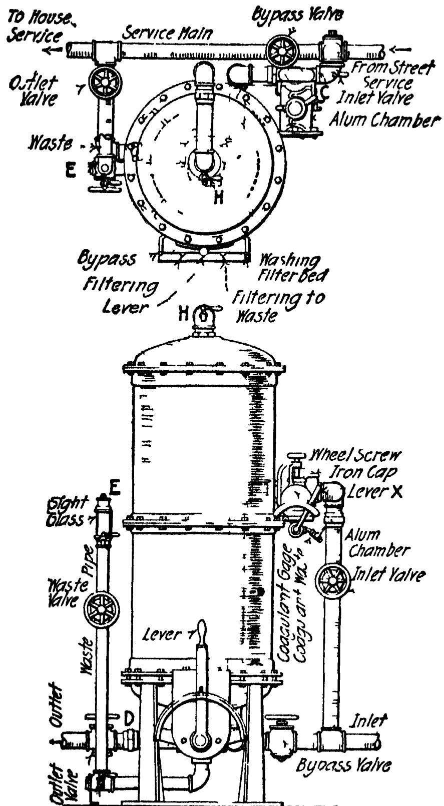
Diam., in	Working pressure 65 lbs per sq in — Shell: Min eff joint, per cent	Shell: Thickness, in	Head† thickness in	Working pressure 100 lbs per sq in — Shell: Min eff joint, per cent	Shell: Thickness, in	Head† thickness in	Working pressure‡ 125 lbs per sq in — Shell: Min eff joint, per cent	Shell: Thickness, in	Head† thickness in
24	50	$\frac{3}{16}$	$\frac{1}{4}$	50	$\frac{3}{16}$	$\frac{1}{4}$	50	$\frac{1}{4}$	$\frac{5}{16}$
30	50	$\frac{3}{16}$	$\frac{1}{4}$	57	$\frac{1}{4}$	$\frac{5}{16}$	50	$\frac{5}{16}$	$\frac{5}{16}$
36	50	$\frac{3}{16}$	$\frac{1}{4}$	57	$\frac{1}{4}$	$\frac{5}{16}$	70	$\frac{1}{4}$	$\frac{3}{8}$
42	57	$\frac{1}{4}$	$\frac{5}{16}$	70	$\frac{1}{4}$	$\frac{3}{8}$	70	$\frac{5}{16}$	$\frac{7}{16}$
48	57	$\frac{1}{4}$	$\frac{5}{16}$	70	$\frac{1}{4}$	$\frac{3}{8}$	70	$\frac{5}{16}$	$\frac{7}{16}$
54	57	$\frac{1}{4}$	$\frac{5}{16}$	70	$\frac{5}{16}$	$\frac{7}{16}$	70	$\frac{3}{8}$	$\frac{1}{2}$
60	57	$\frac{1}{4}$	$\frac{3}{8}$	70	$\frac{5}{16}$	$\frac{7}{16}$	67	$\frac{7}{16}$	$\frac{1}{2}$
72	72	$\frac{1}{4}$	$\frac{3}{8}$	69	$\frac{3}{8}$	$\frac{1}{2}$	66	$\frac{1}{2}$	$\frac{9}{16}$
84	70	$\frac{5}{16}$	$\frac{7}{16}$	66	$\frac{1}{2}$	$\frac{9}{16}$	66	$\frac{9}{16}$	$\frac{11}{16}$
96	69	$\frac{3}{8}$	$\frac{7}{16}$	68	$\frac{1}{2}$	$\frac{5}{8}$	68	$\frac{5}{8}$	$\frac{3}{4}$

Standard manholes 11 by 15 in or 10 by 16 in
† Heads dished to radius of diameter of tank
‡ Hydrostatic test 50 per cent in excess of working pressure

must precede filtration, as is usually the case, a pressure filter is rarely more economical either in construction or operation, and it is generally more difficult to get satisfactory results with it than with gravity filters.

Specifications, Cast Iron Pressure Filters.*

To be gray iron casting having a tensile strength of approximately 20,000 pounds per square inch. Hydrostatic test 50 per cent in excess of working pressure to be applied Heads dished to radius equal to diameter of



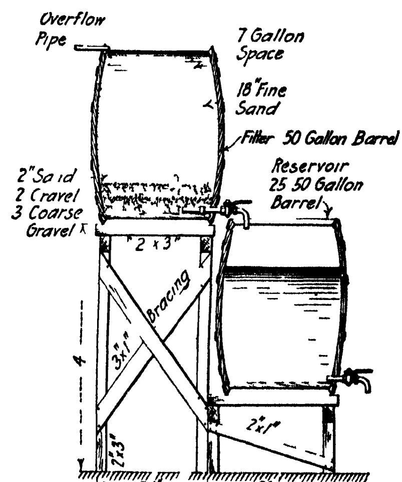


FIG. 293. (Small filters) FIG 294

FIG. 293 —Front elevation and plan of single-cylinder water filter showing, by shaded lines, suggested by-pass arrangement and connections to be made to supply house and water mains

(Loomis-Manning)

FIG. 294.—Intermittent barrel filter for drinking water For contractors' camps, country use, etc.

shell may be modified with rib reinforcement to same thickness as shells. Variations of ⅛ in. in these thicknesses of shells and heads and flanges to be permissible.

Small filters are usually of the pressure type (pp 717). They are, as a rule, fitted with a differential device for feeding alum (not sulfate of alumina),

* Recommended by the Associated Manufacturers of Water Purifying Equipment, July 7, 1922.

(Fig. 293). An intermittent sand filter and reservoir made of 50-gal. barrels is shown in Fig. 294. Water is applied at the rate of three buckets or garden pots at a time. The sand should be allowed to drain after each application. The filter has a capacity of 150 gal. per 24 hr.

Table 265. Dimensions of Cast-Iron Pressure Filters*

Diam in	65-pounds working pressure		100-pound working pressure	
	Shell thickness, in	Head and flange thickness, in	Shell thickness, in	Head and flange thickness, in
12	$\frac{5}{8}$	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{7}{8}$
14	$\frac{5}{8}$	$\frac{7}{8}$	$\frac{11}{16}$	$\frac{15}{16}$
16	$\frac{5}{8}$	$\frac{7}{8}$	$\frac{11}{16}$	$\frac{15}{16}$
20	$\frac{11}{16}$	$\frac{15}{16}$	$\frac{3}{4}$	1
24	$\frac{11}{16}$	$\frac{15}{16}$	$\frac{13}{16}$	$1\frac{1}{16}$
30	$\frac{3}{4}$	1	$\frac{7}{8}$	$1\frac{1}{8}$
36	$\frac{13}{16}$	$1\frac{1}{16}$	$\frac{15}{16}$	$1\frac{3}{16}$
42	$\frac{13}{16}$	$1\frac{1}{16}$	1	$1\frac{1}{4}$
48	$\frac{7}{8}$	$1\frac{1}{8}$	$1\frac{1}{16}$	$1\frac{5}{16}$

* Tensile strength of steel plate 55,000 lbs to 65,000 lbs

Gravity Filters. Gravity filters are built of wood or reinforced concrete, usually the latter; it is of paramount importance that the concrete work be watertight to prevent the mixing of unfiltered and filtered water and the entrance of air into the underdrains. The arrangement of a typical rapid filter plant is shown in Fig. 295. Ordinarily rapid filters are arranged in double rows with conduits, pipes, and appurtenances in a gallery between. Typical arrangements are shown by Figs. 295 and 296.

Filter Sand.* Because it is graded hydraulically during washing, sand for rapid filters must be carefully selected. Its effective size may vary from 0.35 to 0.50 mm., although a range of from 0.40 to 0.45 suits most conditions best. If used in a softening plant, the sand grains will increase in size due to accretions of carbonate unless the water be acidified before filtration. The uniformity coefficient of rapid filter sand should not exceed 1.6, and it should contain not over 2 per cent. of particles passing a No. 80 sieve (separation size 0.25 mm.), or, expressed more in accordance with the needs of filters washed at high velocity, it is desirable that in the portion of the sand which will pass a No. 60 sieve (separation size 0.36 mm.), the 1 per cent., 10 per cent., and 60 per cent.-size should be determined and preference should be given to sands which most nearly approach the condition where the 1 per cent.-size is not less than 1.85 times the 10 per cent.-size, and the 60 per cent.-size is not greater than 1.5 times the 10 per cent.-size. The tendency of practice is toward better treatment, coarser sand, and higher rates of washing.

Areas and Rates. Nominally, rates may vary from 100 to 150 mgad., but ordinarily areas are provided for rates of 2 gal. per sq. ft. per min. plus an allowance for wash water, say an area of 365 sq. ft. per mgd. At Detroit, higher rates are employed equivalent to 265 sq. ft. per mgd. With better preliminary treatment and more general use of chlorine as a disinfectant, there is a tendency toward higher rates and coarser sands, but the most

* For general sand data, see p 703
† With carbon dioxid.

economical rate of filtration to be adopted in any case is dependent upon the character of the applied water. Owing to high initial loss of head and consequent narrow ranges in losses of head possible, the employment of rates higher

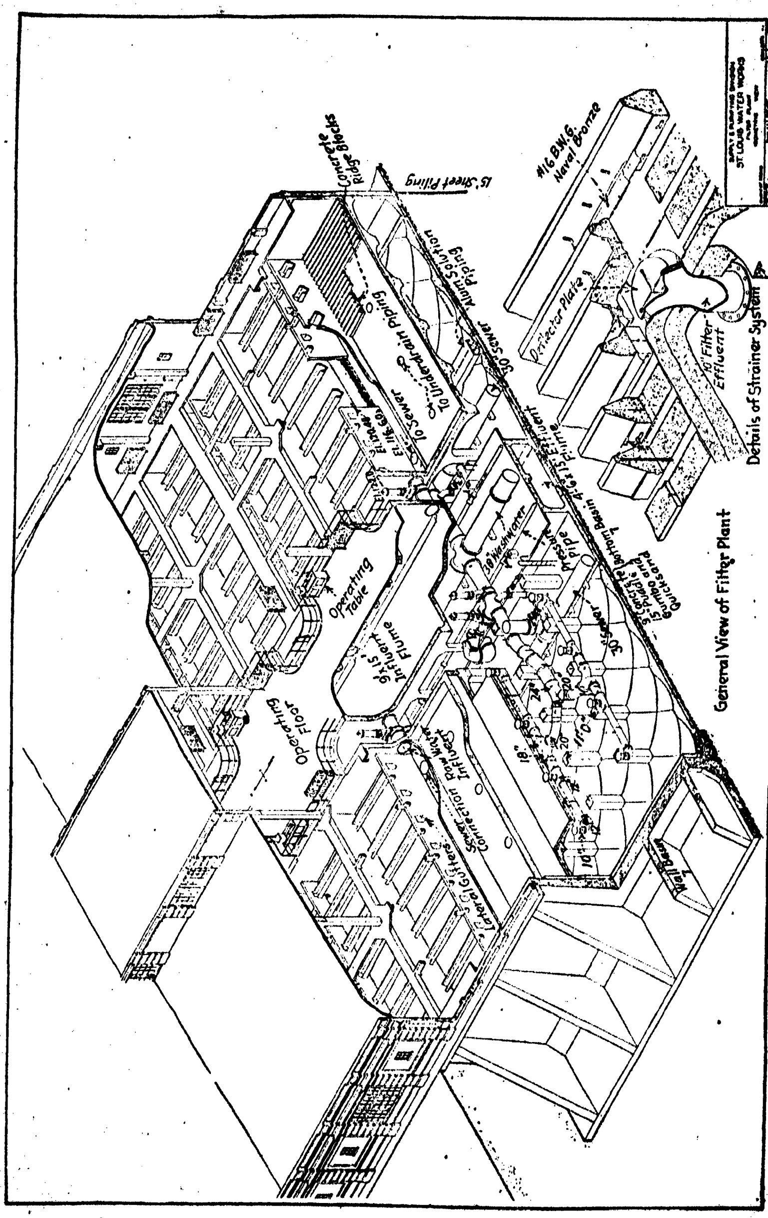
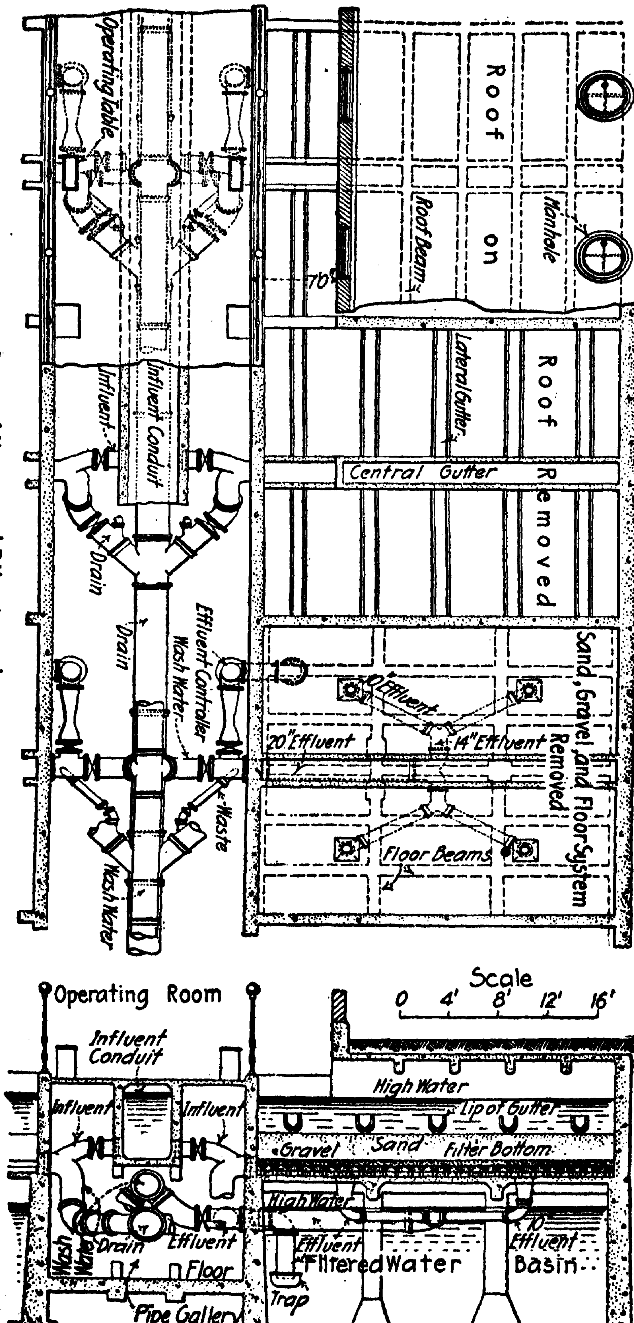


FIG. 295.—Diagram of typical rapid filter. (St. Louis, Annual Report, Water Commissioners, 1915.)

than 150 mgad. with sands of effective sizes finer than 0.40 mm. is usually impracticable. Lower rates may be indicated when waters are difficult to coagulate, or contain much suspended matter or color or are highly polluted



Plan of Mechanical Filter (Akron)

Cross Section Through Mechanical Filter (Akron)

FIG. 296.—Akron, O., filter plant.
(Courtesy of F. A. Barbour, C. E.)

bacteriologically. A considerable factor of safety must be used in filter design, especially where the character of the raw water fluctuates greatly.

Table 266. Sizes and Thickness of Gravel in Rapid Filters

Place	Cincinnati	Newmarket, N. H.	Providence	Miraflores C. Z.	Toledo
Type of bottom	Strainers and troughs	Wheeler	Harrisburg	False bottom	Perforated pipes
Thickness of bottom layer, in.	2″	3″	6″	8″	5″
Size of bottom layer, in.	1″–2″	1″–1½″	1″–2″	1″–1¾″	1½″–2½″
Thickness of 2nd layer, in.	2″	3″	3″	12″	3″
Size of 2nd layer, in.	¾″–1″	½″–1″	½″–1″	7/16″–1″	¾″–1½″
Thickness of 3rd layer, in.	3″	3″	3″	4″	3
Size of 3rd layer, in.	½″–¾″	3/16″–½″	¼″–½″	3/16″–7/16″	⅜″–¾″
Thickness of 4th layer, in.	4″	3″	3″	....	3
Size of 4th layer, in.	¼″–½″	12 mesh–3/16″	⅛–¼″	....	3/16″–⅜″
Thickness of 5th layer, in.	3″	....	3″	....	3″
Size of 5th layer, in.	Less than ¼″	....	12 mesh–⅛″	....	10 mesh–3/16″
Thickness of 6th layer, in.	....	....	....	....	....
Size of 6th layer, in.	....	....	....	....	....
Total thickness	14″	12″	18″	24″	17″

Place	Minneapolis	Louisville	Columbus	Grand Rapids
Type of bottom	Ridge block & strainer	Manifold strainers	Strainers	Strainers
Thickness of bottom layer, in.	2¼	5″	5″	7.5″
Size of bottom layer, in.	1½″–2″	¾″–1½″	1½″–2½″	1′–2″
Thickness of 2nd layer, in.	3″	4″	3″	1.3″
Size of 2nd layer, in.	1″–1½″	⅜″–¾″	1″–1½″	⅝″–1″
Thickness of 3rd layer, in.	3″	3″	2″	1.1″
Size of 3rd layer, in.	⅝″–1″	3/16″–⅜″	½″–1″	⅜″–⅝″
Thickness of 4th layer, in.	2¼″	2″	2″	1.1″
Size of 4th layer, in.	⅜″–⅝″	10 mesh–3/16″	¼″–½″	5/16″–⅜″
Thickness of 5th layer, in.	2″	....	1½″	3.0″
Size of 5th layer, in.	5/16″–⅜″	....	⅛″–¼″	10 mesh–5/16″
Thickness of 6th layer, in.	2″	....	2½″	....
Size of 6th layer, in.	10 mesh–5/16″	....	Fine pea gravel	....
Total thickness	15″	14″	16″	14″

Filter gravel should be carefully graded into from three to five layers.

The strainer system, underdrain system, or filter bottom serves for removal of filtered water and introduction of wash water. The friction of the system, when filtering, must not exceed 25 per cent. of the friction resistance of the sand, when the filter is first put into service. It is essential that the rate of filtration be uniform throughout the whole sand layer, and that the wash water be distributed as uniformly as practicable over the whole filter area so that it may rise as a plane or layer for the purpose of separating the accumulated coagulant and fine suspended matter without at the same time causing any considerable loss of sand. In the existing successful filters, there is somewhere a plane of maximum resistance which effects this result.

Typical strainer systems are shown in Fig. 297. The most generally satisfactory systems are: (1) manifolds or false bottoms with bronze strainers and a thick layer of graded gravel (14 to 18 in.); (2) the Harrisburg system of perforated pipes with a thick layer of graded gravel (14 to 18 in.); (3) troughs and strainers at the bottoms surmounted with 14 in. of graded gravel (12 to 16 in.); (4) the Wheeler filter bottom with 7 to 12 in. of gravel.

System 2 is the simplest but most liable to corrode; it is often difficult to correct for varying velocity heads in the laterals, although at New Orleans

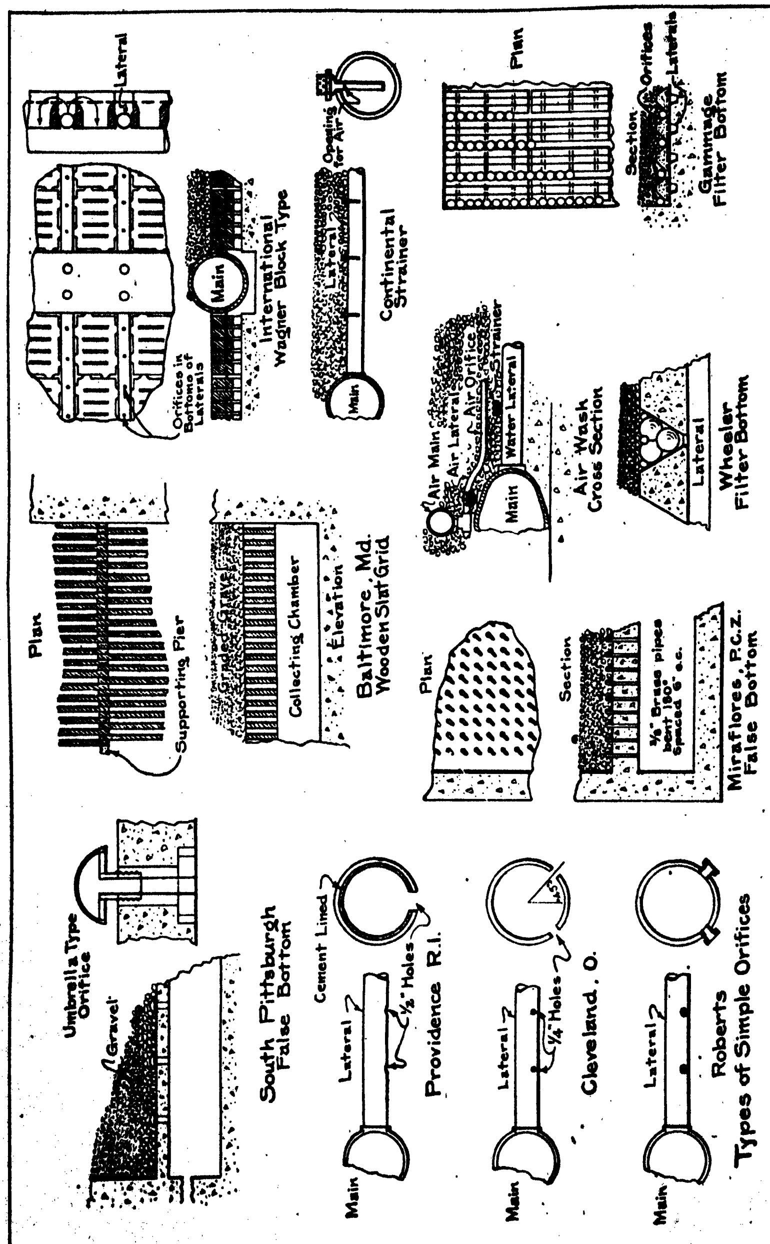


FIG. 297.—Typical filter bottoms.

this is being accomplished by openings of special construction. System 1 exposes the minimum of metal, but in large filters must be carefully designed

to preserve the hydraulic balance in the bed above. System 3, on account of cost, is showing decreased use. System 4 possesses the advantage of uniform material uniformly placed and gives excellent distribution. Strainers in systems 1 and 3 should be of bronze, which increases the cost. The piping in system 2 may be cement-lined.

Wheeler Bottom. System 4, the Wheeler filter bottom used at Akron, Ohio, is patented for protection. It is constructed entirely of concrete with the exception of the orifice tube, which is of brass. In place of strainers, a series of five 3-in. balls, surmounted by one $1\frac{3}{8}$-in. and eight $1\frac{1}{4}$-in. balls in turn surmounted by a layer of graded gravel from 3 to 6 in. thick, is used to retain the sand. The advantages consist in the absence of metal (with the exception of the short brass tube at the apex of the pyramid); the nearly perfect distribution of the wash water secured by the ball-nozzle effect of the balls; the lower cost, and the thinner gravel layer. It was found in some cases, however, that after some years' use the cement spheres became considerably worn down, reducing the efficiency of operation. To remedy this, glazed earthenware spheres have replaced the cement ones at a few plants.

*The Gammage bottom,** entirely of concrete, consists of blocks with opposite orifices fed from laterals beneath. The blocks form ridges between which are placed spheres of concrete. The whole is covered with graded gravel. Orifices are depended upon for distribution while the opposing jets neutralize the velocity head.

Strainer Orifices. Where the area of the orifices compared with the underdrains is unduly large, there is a tendency for the sand to be washed out at one place, thus creating a path of least resistance, while in other portions of the bed the clogging is excessive. Continuance of this unequal distribution of wash water produces mud balls and clogged sections, and also greatly reduces the efficiency of the plant. Too coarse gravel in bottom of filter furthers lateral communication and multiplies this trouble.[7] There is a tendency to increase the size of the orifices and to depend more upon the gravel layer for distribution. The International bottom with pipes having large perforations and with pre-cast concrete blocks between (Fig. 297) is an example of this, but the extreme is the arrangement of 1 by 6 in. boards spaced 1 in. apart, used by Armstrong at Baltimore.

Sometimes the upper layer of gravel is cemented in place, using from 12 to 15 parts of gravel to 1 part of cement. This practice is quite common in Ontario.[7] It is advocated to prevent unequal washing and disturbance of sand layer. So far these layers have kept clean.

At Sacramento, a modification of the Harrisburg system was thoroughly investigated, and the following conclusions reached:

1. Ratio of length of lateral to its diameter should not exceed 60.
2. Diam. of perforations in lateral should be between $\frac{1}{4}$ in. and $\frac{1}{2}$ in.
3. Spacing of perforations along lateral may vary from 3 in. for a diam. of perforation of $\frac{1}{4}$ in., to 8 in. for a diameter of perforation of $\frac{1}{2}$ in.
4. Ratio of total area of perforations in underdrain system to total cross-sectional area of laterals should not exceed 0.5 for a diam. of perforation of $\frac{1}{2}$-in., and should decrease to 0.25 for a diameter of perforation of $\frac{1}{4}$ in.

* Patented.

5. Ratio of total area of perforations in underdrain system to entire filter area may be as low as 0.002, or 0.3 sq. in. per sq. ft. of filter.

6. Spacing of laterals may be as great as 12 in. for satisfactory diffusion, but is limited by total head available.

7. Rate of washing may be varied from 6 to 36 in. per min. (0.5 to 3.0 cu. ft. per sq. ft. of filter per min.) provided the foregoing factors are used in design.

Losses of Head. The total loss of head in rapid sand filters is comprised of the velocity head (*i.e.*, the head required to maintain the velocity of flow), and the component losses of head required to overcome the frictional resistance of: (*a*) the matter accumulated by the filter; (*b*) the sand layer; (*c*) the gravel layer; (*d*) the underdrain system. The initial loss of head of a clean filter is usually not greater than 1.5 to 2.5 ft., and the added loss of head due to the material retained by the filter may reach 10 ft. for a clogged filter, with a total loss of head of 11 to 12 ft.

The maximum loss of head obtainable depends on the size of filter sand and the strength of the film bridged between the particles. These factors should be considered in design. The economical size of sand is that which will give a maximum loss of head slightly in excess of the available loss of head. With better washing and better coagulation, the tendency is toward the use of sand of 0.50 mm. effective size or even more. Because of pumping costs, maximum losses of over 10 ft. are rarely practicable.

Washing. The most efficient washing is accomplished by *wash-water velocities* of 1.5 or more vertical ft. per min., depending upon size of sand grains. Where lesser rates are employed, the use of air or water jets, or some mechanical device for agitating the sand is necessary for best results. Wash water should be applied gradually at first and shut off gradually when washing is finished, to avoid mixing sand and gravel. Filters should not be washed too clean; otherwise low efficiency will occur right after washing. With non-uniform sand washed at high velocity, the surface of the bed after washing will be composed of very fine particles which may form a layer so dense as greatly to decrease the period between washings. This condition may be overcome, in ordinary cases, by scraping off the fine layer after washing; in extreme cases, by so manipulating the wash-water valve that stratification near the surface will not be marked. When the filter clogs frequently, the wash water may be applied for a short period just to lift the upper portion of the sand bed and without wasting any of the wash water. Agitation with air, or with jets of water forced through a special system of pipes, may also be used to break up the accumulation at the surface of the bed.

Optimum Velocity. The sand in mechanical filters, after washing, should be stratified in layers according to the hydraulic values* of the particles, as shown in Fig. 282. This result cannot be obtained unless the sand be thoroughly floated during washing. The degrees of expansion of five different sands for different velocities of wash water, are shown in Fig. 299. In studying the relation between *washing velocities* and *filter sand*, the 60-per cent. separation size is of more value that the "effective size" (10-per cent. separation size). Relation between *sizes of sand particles* and *per cent. of expansion of sand layer*, for normal sands, at optimum velocity of wash water is shown in

* See footnote, p. 705.

Fig. 301. The relation between the upward velocity of wash water and loss of head due to various depths of filter sand is shown in Fig. 298. Figures 298, 299, 300, 301 may be used for wash-water estimates. They are based on

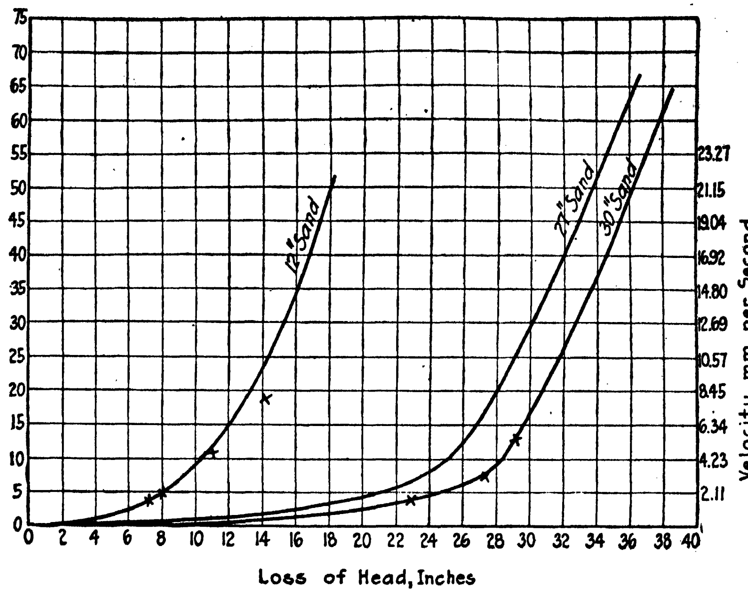


Fig. 298.—Relation between upward velocity of wash-water and loss of head in usual filter sands.

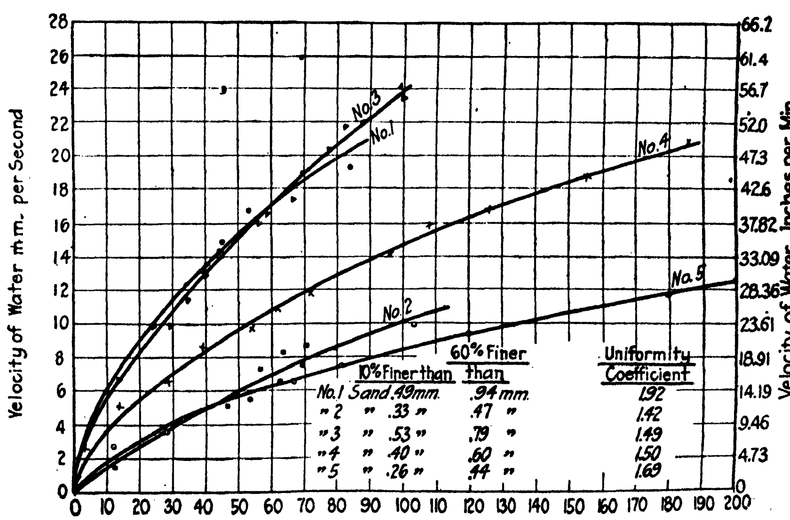


Fig. 299.—Upward velocity of wash water and expansion of sand in mechanical filter. (No air used.)

tests with the Wheeler filter bottom, which gives more nearly uniform distribution of wash water than some other types. This fact must be considered when applying these data to other strainer systems.

Example: To determine optimum velocity of wash water and percentage expansion at optimum velocity of a sand having particles 10 per cent. finer than 0.40 mm. and 60 per cent. finer than 0.65 mm. In Fig. 300, the optimum velocity is read from the 60 per cent. curve as 10.4 mm. per sec. (or 24.6 in. per min.).

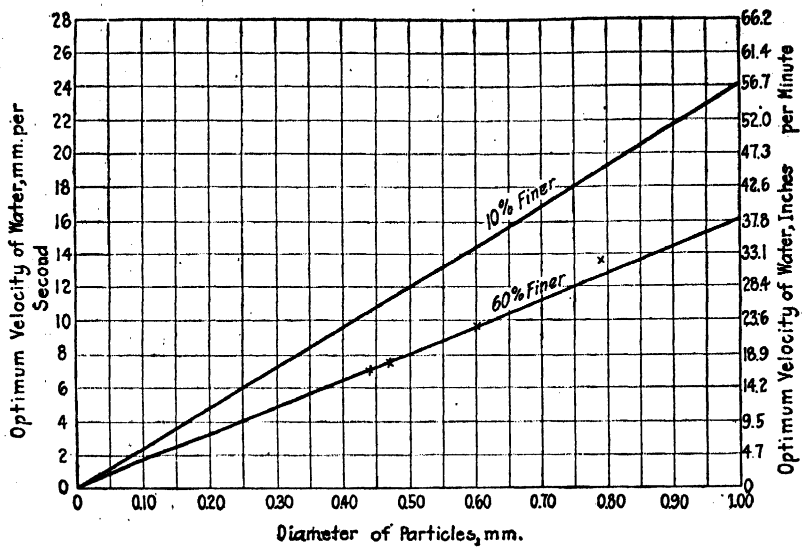


FIG. 300.—Optimum velocities of wash water for sands of various sizes. (Ten per cent. curve is derived from 60 per cent. curve, assuming uniformity coefficient of normal sand to be 1.50.)

In Fig. 301, the degree of expansion is read from the 60-per cent. curve as 50 per cent. The wash-water overflow* for a sand layer 30 in. thick should, therefore, to

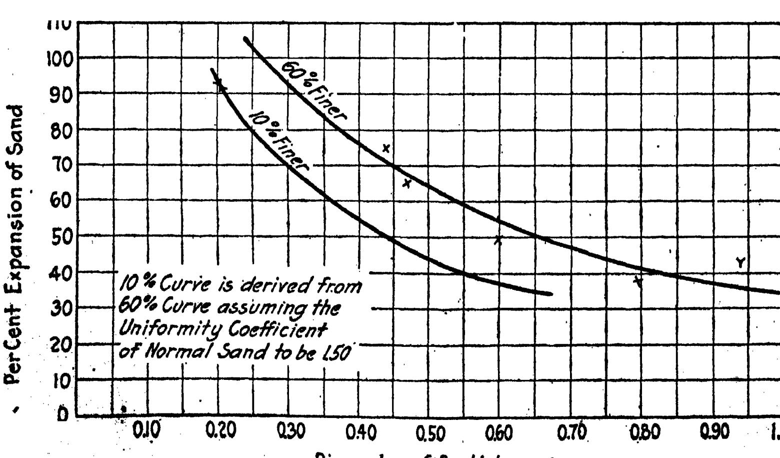


FIG. 301.—Expansions of normal sands at optimum velocities of wash water.

minimize loss of sand, be placed more than 15 in. above the sand surface. Ordinarily the 60-per cent. curve is safe, but 10-per cent. curve may be used when

* Sometimes termed "trough" and "gutter."

uniformity coefficient is 1.5. With a given velocity of wash water, sands with rounded grains expand less than others (Fig. 299).

Frequency of washing depends upon character of raw water and degree of removal of suspended matter effected in the coagulating basins. A filter is usually washed when the loss of head reaches a certain maximum, 4 to 10 ft., usually once in 4 to 24 hr. The coagulum will "break" through in some filters operating with some waters before the maximum loss of head is reached; such filters should be washed immediately, or before breaking.

Duration of washing is usually 4 to 8 min. and the *period out of service for washing* is usually 8 to 15 min.

Wash water for mechanical filters is best supplied by gravity from an elevated tank. For washing at high velocity, say 24 in. per min., the head at the strainer should be about 12 ft. plus the head required for velocity and to overcome friction head of the underdrains; for low velocity washing the head should be less. Where air is employed for washing, it is best supplied from a gas-holder, although a pressure blower* may be used. In lieu of storage of air and water in gas-holders and tanks, wash-water pumps and air blowers may be used. At Montreal, air is supplied from a gasometer, and water from a pump. Pumps should be of a size ample to meet all requirements. Excessive pressures should be carefully avoided. Eductors† connected both with high pressure mains and draining from the filtered water basin have been used at Paris, Ky., and Zelienople, Pa.

Quantity of Wash Water. On an average, 0.5 to 7 per cent. of filtered water is required, but in cases of frequent clogging as much as 15 per cent. is used. Wash water should be provided for maximum, not average, conditions.

Waste Wash Water Disposal. It is often necessary to remove the sludge from waste wash water before discharge into a stream. In rare cases the clarified water is returned in part to the entering raw water. For this purpose ordinary subsidence may be used and the sludge discharged into lagoons or carried away. The Dorr thickener may be used to dewater the sludge partially. In general the methods follow those used for sewage treatment.

Loading. The sand bed in a rapid filter has a certain functional capacity for efficient purification, dependent upon several factors such as color, bacteriological content, turbidity, thoroughness of treatment, of applied water; rate of filtration, size and depth of sand, and efficiency of washing.

Because color removal is dependent upon adsorption, the mass of flocs formed by coagulation and not the films around the sand on a rapid filter are of chief importance. Consequently, properly coagulated water may be successfully decolorized under a wide range of filter conditions.

The *turbidity* of water which may be applied successfully to filters at normal rates (125 mgad.), may reach 50 p.p.m., but in general it is desirable to maintain the turbidity below 20, preferably below 15 p.p.m. With increase of weight or size of sand the possibilities in this direction diminish rapidly.

The same laws apply to bacterial loading as to loading with turbidity. The limits of bacterial loading have been studied for Ohio River plants by the U. S. Public Health Service, which found the results given in Table 267.

* Roots, Connersville and others.
† Chaplin-Fulton: Schütte & Koerting; Watson & McDaniel.

Table 267. Average Numbers of Bacteria Observed in Applied Water, with Corresponding Numbers Observed in Unchlorinated Effluent

Ohio River Plants Employing Single Coagulation and Subsidence

Applied water range per c.c.	Number of tests	Average bacterial count per c.c.		Per cent. of applied water count	
		Applied water	Filter effluent	Remaining	Removal
Agar, 24 Hrs., 37°C.					
0– 100	1326	47	4	9.0	91.0
101– 250	321	177	14	8.2	91.8
251– 500	393	360	38	10.7	89.3
501–1000	132	735	95	12.3	87.7
Over 1000	101	2060	290	13.3	86.9
			Mean......	10.7	89.3
Agar, 48 Hrs., 20°C.					
0– 100	248	74	12	18.4	81.6
101– 250	319	184	28	15.9	84.1
251– 500	386	383	59	15.4	84.6
501–1000	409	803	180	21.6	78.4
1001–2000	404	1450	214	14.3	85.7
Over 2000	180	3280	508	16.4	83.6
			Mean......	17.0	83.0

Bact. Coli Index

Applied water index range per 100 c.c.	Number of tests	Average Bact. coli index per 100 c.c.		Per cent. of applied water count	
		Applied water	Filter effluent	Remaining	Removed
0– 10	134	10	2.6	26.0	74.0
10– 100	339	100	3.9	3.9	96.1
100– 1000	256	1000	4.6	0.46	99.54
1000–10000	63	10000	4.9	0.05	99.95

Table 268. Average Numbers of Bacteria Observed in Applied Water, with Corresponding Numbers Observed in Unchlorinated Effluent at a Rapid Sand Filtration Plant, Niagara River

Applied water count range per c.c.	Average bacterial count per c.c.		Per cent. of applied water count	
	Applied water	Filter effluent	Remaining	Removed
Agar, 48 Hrs., 20°C.				
0–2700	2240	790	35.3	64.7
2701–4000	3400	1100	29.4	70.6
4001–5500	4720	1360	28.8	71.2
Over 5500	7350	2300	31.3	68.7
		Mean.........	31.2	68.8

Bact. Coli Index

Applied water index range per 100 c.c.	Average Bact. coli Index per 100 c.c.		Per cent. of applied water count	
	Applied water	Filter effluent	Remaining	Removed
0– 500	330	38	11.5	88.5
500–1000	860	55	6.4	93.6
1000–5000	3780	71	1.9	98.1
Over 5000	6850	83	1.2	98.8

Results for a plant taking water from the Niagara River are given in Table 268. Comparing the two tables it will be noticed that the filters treating the clearer Niagara water show lower efficiency (68.8 per cent.) as compared with 83.0 per cent. These efficiencies may vary widely for short periods but tend to remain constant for periods of a day or more.

There is a close relation between the turbidity of the applied water and the period of service (length of run), and between the turbidity and the percentage of wash water. Again the structural strength of the coagulated matter, that is, its resistance to breaking, determines the limit of loss of head which may be reached before washing is necessary. Usually coagulating basins are more economical than filters for removing the bulk of turbidity.

With the use of chlorine, there is a tendency to sacrifice clearness of effluent which without disinfection varies almost directly with the bacteriological efficiency.

Properly loaded filters should deliver a clear effluent for a period of 8 hr. or more; they should decolorize the properly coagulated water to less than 10 p.p.m. and should so reduce the bacteria that the effluent may be completely disinfected with doses of chlorine not large enough to cause objectionable tastes.

ACCESSORIES

Rate Controllers. Next to proper coagulation, efficiency of filtration is dependent on a uniform rate of filtration. While rapid filters may be controlled

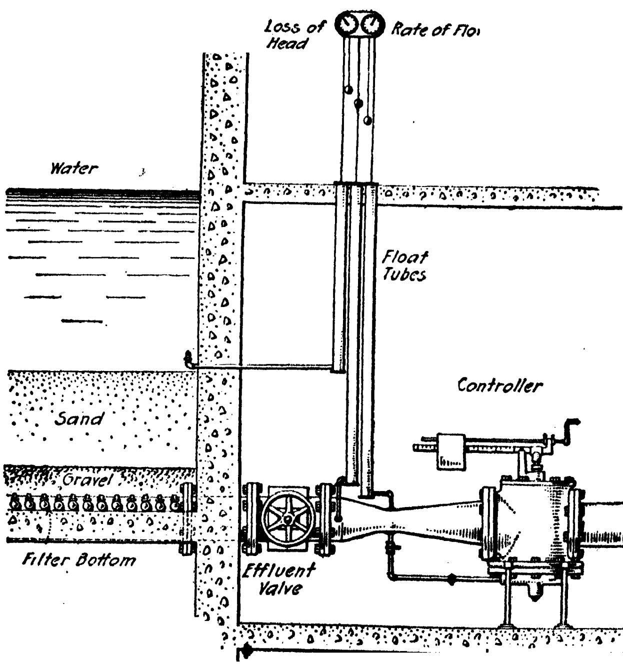
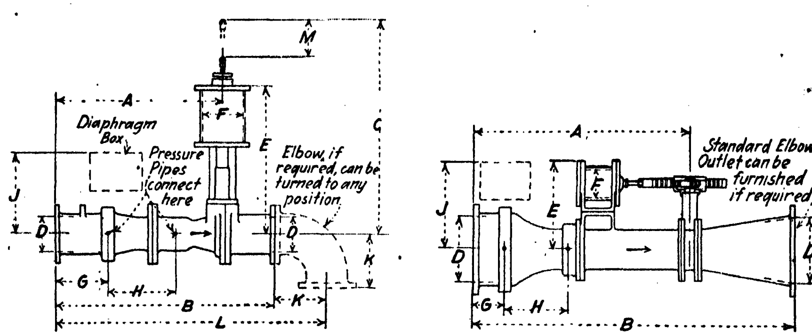


FIG. 302.—Rate controller with combined loss-of-head and rate-of-flow gage for gravity filter. (Simplex.)

by hand, it is better, often essential, to use automatic controllers or modules. Figure 302 illustrates a rate controller of the velocity type, combined with

rate of flow and loss-of-head gages; Fig. 303, employing an hydraulically operated valve with compensating diaphragm. Figure 303 shows two types of Venturi controller tubes with hydraulic and rotating valves. Other well-known controllers are the Vivian, Earl, and Pittsburgh.

Controllers on individual filters may be governed by a master controller, such as that made by the Builders Iron Foundry, Providence, R. I. The device slows down all filters simultaneously as the filtered water basin becomes full, and *vice versa*. There is another type which changes filter rates at will



Design A Design B

D	Million of gallons per 24 hr.*			Dimensions in inches											Approx. weight, lb.
	Min.	Normal	Max.	*A*	*B*	*C*	*E*	*F*	*G*	*H*	*J*	*K*	*L*	*M*	
							Design A								
5	.120	.25	.375	11¼	34	38	26¼	8	11¼	9⅜	14	7½	41½	5⅝	400
6	.225	.45	.675	32⅞	40	42	27⅝	8	11½	11¼	16¼	8	48	7⅛	450
8	.375	.75	1.125	38	50	49	33$\frac{13}{16}$	10	12	15	18¼	9	59	8½	700
10	.500	1.00	1.500	46¾	60	57	39⅛	10	12½	18¾	18½	11	71	10¾	1100
12	.740	1.50	2.250	54¼	70	65	44¾	10	13	22½	18¾	12	82	12¾	1550
							Design B								
14	1.000	2.00	3.000	47¼	70	....	24½	8	7	14	20	....	....	....	1600
16	1.250	2.50	3.750	54	80	....	25½	8	8	16	21	....	....	....	2200
18	1.625	3.25	4.875	60¾	90	....	28	8	9	18	22	....	....	....	2700
20	2.000	4.00	6.000	67½	100	....	29	10	10	20	23	....	....	....	3500
24	2.875	5.75	8.625	81	120	....	30	10	12	24	25	....	....	....	5000
30	4.500	9.00	13.500	101¼	150	....	34½	12	15	30	28	....	....	....	7800

* The listed range of capacity (minimum to maximum), is for standard self-contained, or diaphragm, type controllers and can be increased somewhat for unusual requirements. The water-column or mercury-column type controllers permit a much wider range. The friction loss at the normal rates listed will not exceed 5 inches of water. Intermediate and additional sizes of controllers are frequently being added, therefore the list is not complete.

FIG. 303.—Venturi effluent rate controller tube. Dimensions and capacities.

from a central point. Control may be hydraulic or electric. If the latter be used, the apparatus should be protected against dampness.

Loss-of-head gages* are of two types—float operated, and manometers. In the former, differential gears enable the difference in elevation to be recorded. In the Simplex gage mercury has been substituted for water in the float tubes. Gages of the manometer type obviate the trouble due to floats

* Electrically operated gages are used at Sacremento; see *E. N. R.*, Aug. 27, 1925, p. 346.

and cords. They operate successfully although they are located above the flow line, and depend upon a vacuum. They must be kept clean and tight.

Rate of flow gages are usually of the Venturi type and are either indicating* or recording or both. They are usually combined with loss-of-head gages.

Operating Tables. In large plants, especially, it is convenient and economical to operate all valves electrically or hydraulically. Controllers for these valves are placed on an operating table which also may carry loss-of-head and rate-of-flow gages, devices for sampling, and other devices. See Fig. 304.

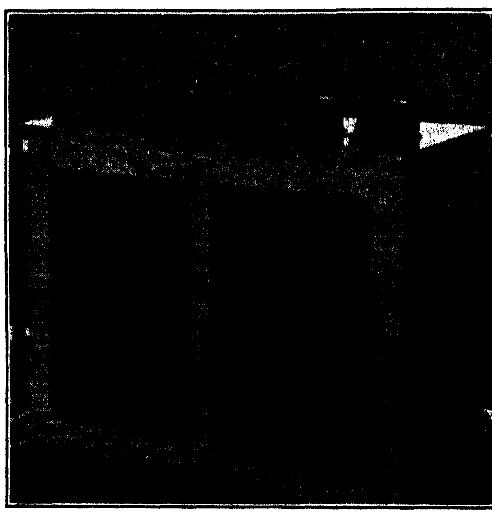


FIG. 304.—Operating table with loss-of-head gage.
(International Filter Co.)

Wash-water troughs* should be of ample size and so spaced that the maximum horizontal path of travel for any particle of wash water shall not exceed 3.5 ft. Assuming a high washing rate, ample and frequent troughs counteract the formation of mud balls.

Calculation of Capacity. P. B. Streander[8] has embodied C. N. Millers' formula† in a diagram, Fig. 305.

OPERATION

The operation of a filter is fully as important as its design.

General inspection of a purification plant is constantly necessary, particularly if the character of the raw water fluctuates rapidly. In slow filters, the condition of the sand is of utmost importance, and in large plants the management of the sand-handling devices requires a great deal of skill.

Rate of filtration may be determined and regulating devices checked by closing the filter inlet and measuring the water filtered by observing the depression of the flow line and computing the volume of water equivalent thereto.

* Also called "wash-water overflow" or "gutter."

† Ellms, "Water Purification," Appendix. (McGraw-Hill Book Company, Inc., 1917). For tests see "The Flow of Water in Wash Water Troughs for Rapid Sand Filters," F. V. Fields, *Cornell Civil Eng.*, Vol. 26, 1918, p. 295.

Filter controllers, loss-of-head gages, and all other automatic devices should be frequently inspected and kept in order; otherwise inferior results will be obtained.

Sand bed accretions,* or growth of filter sand, are common phenomena in rapid filters. Figure 306 from paper by Baylis[9] pictures this.

Considerable study has been given to this problem but no adequate preventive has yet been found. These accretions or "mud balls" must be removed by raking during washing, by hydraulic jetting or by scraping.

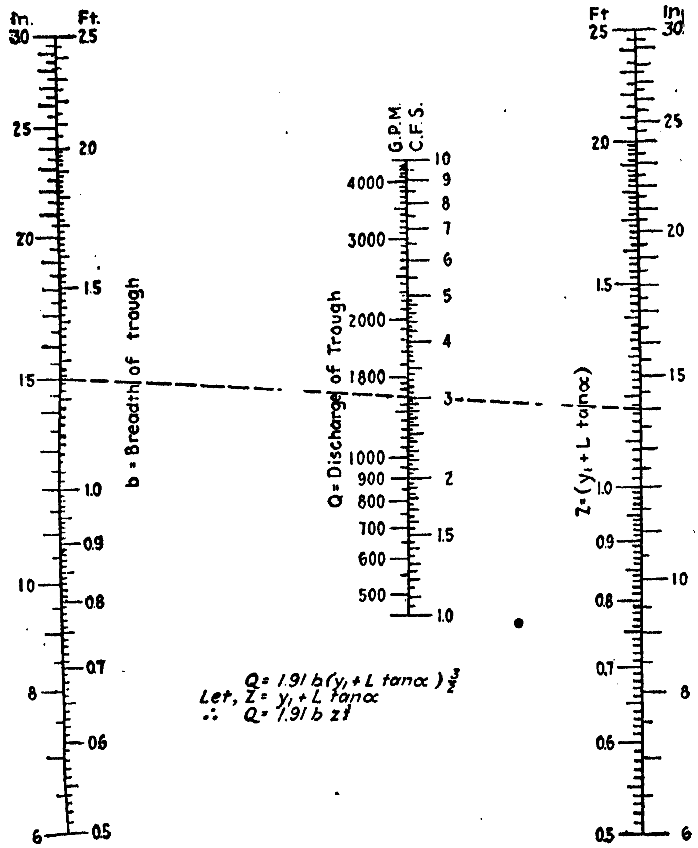


FIG. 305.—Capacities of square wash-water troughs.

Having the total discharge capacity of the trough in gallons per minute or cubic feet per second on scale 3, project from this point to breadth of trough assumed on scale 1 and obtain the value of Z on scale 2. The dimensions of B and Z are for rectangular shape troughs. For shapes of troughs not rectangular application of the above chart can be made by assuming a rectangular section of equivalent area. The value of Y_2 or the depth of water at the discharge end of the trough applies to a case where the discharge is into a chamber in which the water level is at or near the level of the water in the lower end of the trough. In case of free fall the actual value will be less owing to the weir action.

Air binding of filters sometimes also causes trouble, the cause being attributed to high concentration of algae causing supersaturation, dissolved oxygen, washing at too great loss of head, or to leaks in the walls of the filters.

* See also, "Filter Underdrain, Sand Bed, and Wash Water Experience," a symposium in *E. N. R.*, Vol. 85, 1920, pp. 934, 984, Vol. 86, 1921, p. 371.

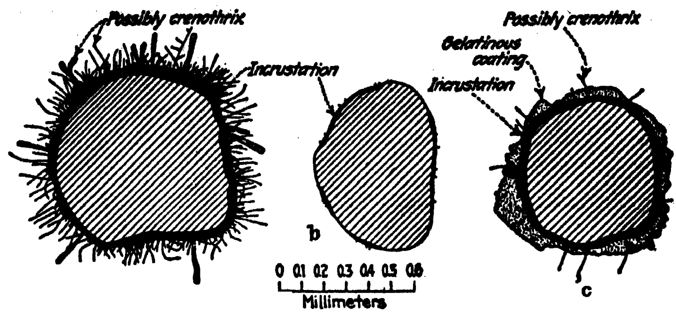


Fig. 306.—Sand grains under camera lucida before and after installing washing jets.

In the part of a filter without jets the sand grains had an incrustation about 20 microns thick (Fig. 306*a*) from which were growing numerous filamentous bacteria. The gelatinous coating (*c*) which gets excessive in September and October, dies after a few weeks and is easily removed by ordinary washing. During this excessive growth period the filamentous organisms are apparently killed. The incrustation is a permanent growth much harder than the temporary outside growth. However, much of it can be removed by surface jet action as noted in (*b*), which is from a portion of a filter in which jets had been operating for five months. This sample was taken at the same time as (*a*).

Table 269. Equivalents of Various Measures of Rate of Filtration

Rate	Million U.S. gals. per acre per 24 hrs.	Million Imperial* gals. per acre per 24 hrs.	U. S. gals. per sq. ft. per hour	Imperial* gals. per sq. ft. per hour	Cu. ft. per sq. yd. per hour	Vertical velocity in in. per hour	Vertical velocity in millimeters per hour	Vertical velocity in meters per 24 hrs., = cu. meters per sq. meter per 24 hrs.
1 million U.S. gals. per acre per 24 hrs	1.0	0.833	0.96	0.80	1.15	1.53	39.	0.935
1 million Imperial gals. per acre per 24 hrs	1.200	1.0	1.15	0.96	1.38	1.84	46.8	1.122
1 U.S. gal. per sq. ft. per hr	1.045	0.870	1.0	0.83	1.20	1.60	40.7	0.978
1 Imperial gal. per sq. ft. per hr.	1.255	1.045	1.20	1.0	1.44	1.92	48.9	1.174
1 cu. ft. per sq. yd. per hr	0.869	0.724	0.83	0.69	1.00	1.33	33.9	0.813
1 linear in., vertical velocity, per hr	0.652	0.543	0.62	0.52	0.75	1.0	25.4	0.610
1 hundred linear millimeters, vertical velocity per hr	2.566	2.139	2.46	2.05	2.95	3.94	100.0	2.400
1 linear meter, vertical velocity per 24 hrs. = 1 cu. meter per sq. meter per 24 hrs	1.069	0.891	1.02	0.85	1.23	1.64	41.7	1.0

Statistics of Operation. Analytical methods adopted by American Public Health Association are recommended (see p. 593). Elimination of unnecessary analytical work is important. Determination of nitrogen in the usual form of albuminoid ammonia, nitrites, and nitrates serves no useful purpose, except in special cases. Some of the simpler physical tests—turbidity, color, odor, microscopical examinations, alkalinity, hydrogen-ion concentration, iron, and carbonic acid—are most valuable. Turbidimeters like those used by Ellms or Baylis should be used for turbidity of effluent. These detect 0.1 p.p.m. of turbidity. In special cases, dissolved oxygen and manganese should be determined. Bacteriological tests are always important.

Many records relating to engineering matters are useful and should be kept. It is not necessary to include all in published reports.

Sampling is as important as are analytical methods. Bodies of water are not homogeneous. *Frequency of sampling* should depend upon frequency of change in character of water examined. This has a limitation which is controlled by practical and financial considerations. Results based on infrequent samples, however, are less valuable than those based upon frequent ones. Irregular sampling gives the most unreliable results.

Grade of Supervision. Control of purification plants may be graded as follows: First-grade plants are those where analyses of filtered water are made one or more times a day, and where engineering and such other data regarding operation of plant as are necessary are collected by one or more attendants constantly employed. Second-grade plants are those where analyses are made regularly, say, once a week or once a month by a trained analyst, and where an attendant constantly on duty makes simple daily tests. Third-grade plants are those where analyses are made irregularly and infrequently, and where no daily tests are made by the attendant.

Standard forms for statistics of operation and efficiency may be purchased from the *N. E. W. W. A.*, Tremont Temple, Boston. The *A. W. W. A.* recommends that the following data be obtained:

(1) Brief description of the sources of supply; (2) general and special characters of the water handled; (3) brief description of the purification system and method of treatment; (4) points of chemical applications; (5) kind of chemical feed devices; (6) rated capacity of works; (7) name of designer of principal features; (8) date works put in operation; (9) subsequent major additions, such as basins and filters; (10) cost of purification works, including what features; (11) available volume in plain subsidence basins, million gal.; (12) available volume in coagulation basins, million gal.; (13) net filter area, sq. ft.; (14) list of references to published descriptions of works; (15) comment on special operating experiences during past year; (16) summaries of operating results on recommended standard forms; (17) typhoid statistics.

Essential operating data of purification. Water handled, mgd: raw water delivered to plant; raw water lost by basin flushing or emptying; treated with chemicals; filtered; used in washing filters; delivered to distribution.

Quality of water: turbidity and color; bacteria; Bact coli; microscopic; chemical; alkalinity and hardness.

Chemicals applied, lb.: alum, iron, lime, soda, and chlorine; copper amount and frequency of treatment.

Filters: number units washed; length of runs; final loss of head; per cent. wash water to water filtered.

Electric power (exclusive of main pumping): for wash water and other pumps, chemical feed water, etc.; lighting.

Service water: for chemical solution, hydraulic valve, hose and flushing, toilet, etc.

Heating: Fuel used—lb.

Costs: chemicals (including unloading); attendance; extra labor; power; lighting; heating; wash water; service water.

Suggested data to be included on Daily Log sheet: (1) rates of flow, raw and filtered (Venturi meter); (2) basin gages; (3) turbidity, raw, settled, applied, filtered; (4) color, raw, filtered; (5) chemical rates . . . Alum, iron, lime, soda, chlorine; (6) settings of chemical orifice or dry feed machine; (7) solution tank gages; (8) weight of chlorine cylinders; (9) actual pounds of chemical used; (10)

residual chlorine test of effluent; (11) filter units washed; (12) wash water used; (13) hours filters in service; (14) electric meter readings; (15) chemicals received into storage; (16) attendants on shift.

Test day is period of 24 hr., during which tests are made. This seems to be the shortest period practical to use as a statistical unit in determining efficiency of purification plants. When more than one observation is made during a test day, results should be combined by averaging to obtain single numbers representative of that day, and the fact stated. Assuming the precision of results of daily sampling to be unity, the precision of results of weekly sampling would be only 0.38; of monthly sampling, 0.18; and of yearly sampling, 0.05; while the precision of hourly sampling would be 4.90.

Period of service is the length of time during which the filter is delivering water. *Period of washing* is the length of time required to stop, wash, and start the filter. *Period of operation* is the total length of time during which the filter is either filtering water or being washed.

Efficiency of Operation. The common method of judging efficiency by percentage removal of bacteria is unsatisfactory for several reasons:

1. Bacteria found in effluent are not all from the raw water. A certain and varying number of them represent growths in the lower part of the filter. Bacteria from this source do not vary in number with the number of bacteria in the raw water, but with the rate of filtration, with disturbances occasioned by the collection of air within the filter, and with other factors of operation.

2. It requires time for water to pass from the point where the raw sample is taken to the point where the filtered-water sample is taken, sometimes several hours, and in the case of a water which changes rapidly it is necessary to make an allowance if a correct comparison is to be obtained.

3. Percentage removal is to a certain extent a function of the number of bacteria in the raw water. The percentage removal is relatively high when the raw water contains large numbers of bacteria and relatively low when the raw water contains few bacteria. One reason is that the bacteria which develop within the filter are a smaller proportion of the total number in the effluent when the number passing the filter is large.

4. Percentage removal does not necessarily vary with the number of bacteria left in the filtered water.

5. Infectiousness does not necessarily vary with the number of bacteria but in most waters there will doubtless be some connection between the two. A water polluted with surface wash would doubtless increase in infectiousness, and the numbers of bacteria would increase with increased stream flow. A water regularly polluted with sewage, other things being equal, would increase in infectiousness as stream flow decreased, that is, as dilution became less, while the numbers of bacteria might decrease. Hence, there seems to be no very definite relation between infectiousness and percentage removal, which percentage varies according to the numbers of bacteria in the raw water.

6. It would be better to use "percentage of bacteria remaining;" the numbers show to better advantage variations in operation of the filter, and the figures are smaller and easier to handle; for example, when two filters, with the same raw water, are operating so as to produce 98 and 99 per cent. "removals,"

the numbers of bacteria remaining in the effluent of the first will be twice as many as in the second. Comparing one filter with an efficiency of 99 per cent. with another of 99.9 per cent., both operating with the same raw water, the effluent of the first contains ten times as many bacteria as the second.

7. A single figure showing percentage removal during a certain time gives no idea of the regularity of operation.

8. A common fault of specifications is the provision that the plant shall show a certain percentage removal when the numbers of bacteria in the raw water are below a certain limit. This specification is inadequate. A good purification plant is one which produces a satisfactory effluent every day and every hour of the day; efficiency should be indicated by the percentage of time during which the plant produced a good effluent.

Table 270. Rules for Significant Figures

Determination	Results to be Reported
Numbers of bacteria.	Ordinarily use two significant figures: never more than three
Percentage of time	To first decimal place
Percentage of Bact. coli tests	To nearest whole number
Chlorine	To first decimal place
Hardness	To nearest whole number
Alkalinity	To nearest whole number
Carbon dioxide	To nearest whole number
Iron	To second decimal place
Oxygen consumed	To first decimal place
Manganese	To second decimal place

Cost of Purification—*Slow Filters.* The elements of cost (in addition to fixed charges) are: cost of scraping, raking, ejecting, washing, transporting, and restoring sand, varying from $0.25 to $1.25 per cu. yd.; cost of pumping; superintendence, attendance, and laboratory; repairs and replacements; miscellaneous clearing; light, heat, etc.; care of grounds.

In the case of *rapid filters* the costs of chemicals and wash water replace the cost of sand handling, and in addition there is the cost of cleaning basins.

In a *softening plant* the cost of treatment* involves more factors which may include the cost of carbonization.

The cost of low-lift *pumping* may vary between $0.02 and $0.30 per m.g. lifted 1 ft. The average additional lift for rapid filters is 15 ft., for slow filters, 10 ft. The minimum head for effective aeration is from 4 to 5 ft.

The average quantity of *wash water* in well-designed plants is about 2 per cent.

* For data for a period of years, see reports of New Orleans and Columbus Water Departments.

Table 271. Data Relating to Typical Slow Filter Plants

	Albany, N. Y.	Pittsburgh, Pa.	Springfield, Mass.	Toronto, Ont.* See also p. 698
Designer	Allen Hazen	Morris Knowles	Hazen and Whipple	Hazen and Whipple
Plant put in operation	1899	1908	1910	1912
Population (1920)	115,000	588,590	129,614	512,812
Total cost	Pumping sta. intake. $ 49,745 Filters and sedimentation basin 324,217 Land, eng. and conduit 125,928 Total $499,890	Bid on final filters, for original 46 acres = $3,303,000*	Filters and sedimentation basin, $321,700¶	Cost exclusive of land Reservoir and wharf $702,183.69 Value of land for resr. 7,937.50 Value of land 85,625.00 Reservoir 45,000.00 Wharf 11,592.50 Additional cost chargeable to slow sand; proportion of boiler room, turbo-generator set, etc. 91,183.66 $943,522.35
Cost per acre	$57,900 (inc. sedimentation basin)	$72,000 (final filters only)	$107,200 (inc. sedimentation basin)	....
Source of supply	Hudson River.	Allegheny River.	Little River.	Lake Ontario.
Rated capacity (gal. per acre per day)	3,000,000†	4,000,000	6,000,000	5,000,000 Imp.
Total cap. of subsiding or coagulating basins (gal.)	14,600,000	120,000,000‖	40,000,000	....
Total cap. of filtered water basin (gal.)	600,000	50,000,000	17,000,000	7,500,000 Imp.
Chemicals used	Bleach and sulfate of alumina occasionally to assist sedimentation	Liquid chlorine.	Sulfate of alumina.	None.
Manner of application of chemicals	Applied to final effluent through orifices by gravity.	Filter effluent by hand.	Applied when necessary to tunnel leading to sedimentation basin.	....
No. of filter units	8‡	56	6	12
Net area of filter surface (acres)	5.6‡	56	3	9.6
Depths of filtering materials (in.)	Gravel (12) Sand (48)	Gravel (12). Sand (24) to (48).	30 to 42	Gravel (12). Sand (3½).
Sizes of filtering material (mm.)	Eff. size 0.31, old, 0.45 in pre-filters. Uniformity coef., 2.3 old, and 1.75 in pre-filters.	Eff. size 0.29. Uniformity coef. 2.	Eff. size 0.25–0.35. Uniformity coef., 3.00.	Eff. size 0.25. Uniformity coef. 2.5.
Cleaning of filter system.	Scraped by hand, removed by ejectors and stored in court. Returned by ejectors.§	Scraped by hand, ejected to washers and placed in idle bed.	Scraped by hand, removed by ejectors and stored in court. Returned by ejectors.	Summer—8 mos. scraped by hand, ejected to court, washed and stored. Returned by ejectors and passed through portable sand washers in filters. Winter—4 mos. scraped by hand and washed in filters in portable sandwashers.
No. of coagulating or subsiding basins	1	3	1	None.
Control of rate of filtration	Handregulation; effluent flows through an orifice on which a constant head is maintained.	Simplex rate controller and by hand.	Hand regulation. Venturi meters and indicators.	Hand regulation. Venturi meters and indicators.

* Filtered water reservoir, $417,000; pumps, $90,000; boilers, $67,000; misc. machinery, $120,000; pumping station and connections, $345,000; intakes, etc. $213,000. (Bids in 1905 for plant as originally built.)
† Without pre-filters. ‡ Sixteen pre-filters also used—built in 1907–1908. Total area, 13,000 sq. ft. § Nichols machine also used. ‖ Includes system of contact filters, p. 696, and A-frame baffles for removing colloidal matter (see p. 644). ¶ Not including land, engineering, or overhead charges.

Table 271. Data Relating to Typical Slow Filter Plants.—(*Continued*)

	Washington, D. C.	Wilmington, Del.	Hartford, Conn.
Designer	Allen Hazen	T. A. Leisen	Water Dept.
Plant put in operation	1905	1910	1922
Population (1920)	437,571	130,000	165,000
Total cost	Pumping sta. $ 183,600 Filters & appurtenances 2,197,000 Filtered water resr. 150,000 Eng., land & misc. 825,700 Total 3,356,300*	Final filters $292,800 Sedimentation basin 175,000 Pumping sta. 308,300	$980,600
Cost per acre	$75,700 (filters and appurtenances)	$146,000 (final filters).	Filters and buildings, 175,000.
Source of supply	Potomac River.	Brandywine Creek.	Surface reservoirs.
Rated capacity (gal. per acre per day)	3,000,000	7,500,000	18,000,000 (Total)
Total cap. of subsiding or coagulating basins (gal.)	Water flows through 3 resrs. having total cap. of 649,000,000.	32,000,000	None.
Total cap. of filtered water basin (gal.)	15,000,000	6,000,000	6,000,000
Chemicals used	Sulfate of alumina occasionally. Chlorine occasionally	Liquid chlorine.	None.
Manner of application of chemicals	Sulfate of alumina applied to aqueduct below Dalecarlia resr. Chlorine applied to effluent from filtered water resr.	Applied to final effluent.	....
No. of filter units	29	6	8
Net area of filter surface (acres)	29	2	4.2
Depths of filtering materials (in.)	Gravel (12). Sand (40).	Gravel (14) Sand (24)	Gravel (12). Sand (36).
Sizes of filtering material (mm.)	Eff. size 0.32, uniformity coef., 1.77	Eff. size 0.23, uniformity coef., 1.83	Eff. size 0.26–0.30.
Cleaning of filter system	Scraped by hand, ejected to washers and returned to idle bed or stored in elevated bins.	Scraped and cleaned without removal from filter, by means of "Blaisdell machine."	Scraped and clean.
No. of coagulating or subsiding basins	3	1	None.
Control of rate of filtration	Hand regulation. Effluent flows through Venturi meters provided with indicators to showrate	Hand regulation. Venturi meters and indicators.	Simplex rate controllers.

* Not including preliminary treatment plant added in 1910 at cost of $23,000.

Table 272. Data Relating to Typical Rapid Filter Plants

	New Orleans, La.	Oklahoma City	St. Louis, Mo.	Trenton, N. J.
Plant put in operation	1909	1923	1915	1914
Designer	George G. Earl	Holway and Bretz	Ed. E. Wall	Johnson and Fuller
Population (1920)	387,600	91,000	794,000 (1923)	119,000
Total cost	$2,515,000*	$350,000	$1,357,000§ (filters and appurtenances)	450,000
Cost per million gal. capacity	$28,000	$21,875	$8,480 (filters only)	$15,000
Source of supply	Mississippi River	N. Canadian River	Mississippi River	Delaware River
Rated capacity (gal. per day)	50,000,000	16,000,000	160,000,000	30,000,000
Total capacity of subsiding or coagulating basins (gal.)	30,000,000†	3,250,000	237,000,000	3,500,000
Total capacity of filtered water basin (gal.)	15,000,000	3,500,000	20,000,000 (Baden) 60,000,000 (Bisseles Pt.) 80,000,000 (Compton)	1,500,000
Chemicals used	Lime, sulfate of iron	Lime, alum, carbon dioxide	Lime, sulfate of iron, sulfate of alumina, liquid chlorine	Sulfate of aluminum Chlorine, soda ash
Manner of application of chemicals	Lime added in mixing reservoirs and sulfate of iron as water passes to coagulating basin	Dry-feed machines by gravity	Lime added in mixing channel, then sulfate of iron, water settled and sulfate of alumina added. Chlorine added to effluent	Alum dry feed. Chlorine and soda ash in solution.
Number of filter units	10	8	40	16
Net area of filter surface (sq. ft.)	14,300	5,728	56,000	10,368
Depths of filtering material (in.)	Sand (36)	Sand (26) Gravel (16)	Gravel (12) Sand (30)	44
Sizes of filtering materials (mm.)	Eff. size 0.32–0.36. Uniformity coef. 1.65	Sand. 42 mm. Gravel 1/10 to 2½ in.	Eff. size 0.31–0.46. Uniformity coef. 1.65	Sand, eff. size, 0.49–0.50. Gravel 1–¼in.
Cleaning of filter systems	Reverse flow of water; velocity 25 in. per min.	High velocity wash	Reverse flow of water Velocity 2.0 ft. per min.	Air and water
Number of subsiding or coagulating basins	4‡	2	9‖	2
Control of rate of filtration	Balanced valve controlled by head on orifice and system of balanced floating cylinders	Builders Iron Foundry controllers	Controllers of the Venturi meter type	Venturi
Wash water used, per cent. of filtered water		4½ per cent.	1.409 per cent.	1–2 per cent.

* Includes cost of pumping station, cost of preliminary investigations, wharf and intake, land and engineering.
† Not including two grit chambers, total cap. 6,600,000 gal.
‡ Also two mixing chambers, 2.5 mg. each.
§ Includes engineering but not real estate. The appraised value of old work incorporated in the new plant is as follows: Land, $52,000. Settling basin, $1,923,000. Coagulant house, $160,000.
‖ Besides grit chamber.

Table 272. Data Relating to Typical Rapid Filter Plants.—(*Continued*)

	Wilmington, Del.	Detroit	Louisville, Ky.	Minneapolis, Minn.
Plant put in operation	1917	1923	1909	1913
Designer	Fuller and McClintock	T. A. Leisen	Chas. Hermany	Hering and Fuller
Population (1920)	112,000	993,739	235,000	395,000‖
Total cost	$130,000	About $4,480,000; plus low-lift pump station, $700,000	Filters, clear water basin and laboratory $1,266,000. Coagulating basin $130,000	$1,597,939.10¶
Cost per million gal. capacity	$10,830	$14,000 without station	$17,000	$20,486
Source of supply	Brandywine Creek	Detroit River	Ohio River	Mississippi River
Rated capacity (gal. per day)	12,000,000	320,000,000	78,000,000	78,000,000
Total capacity of subsiding or coagulating basins (gal.)	1,000,000	30,000,000	12,000,000‡	7,400,000**
Total capacity of filtered water basin (gal)	500,000	37,000,000	25,000,000	45,000,000
Chemicals used	Lime, sulfate of alumina, liquid chlorine	Sulfate of alumina	Sulfate of alumina, liquid chlorine	Sulfate of alumina, liquid chlorine
Manner of application of chemicals	Alum solution at entrance of flumeway to coagulating basins. Lime to same flumeway a short interval after adding alum.* Liquid chlorine to effluent.	Dry-feed machines	Solution feed. Chemical added to coagulant basin through constant head orifice	Sulfate of alumina at head of mix chamber, chlorine at effluent from filters
Number of filter units	6	80	18	16–24
Net area of filter surface (sq. ft.)	4,400	108,800	26,000	17,800
Depths of filtering material (in.)	Gravel (18) Sand (24)	Gravel (17) Sand (24)	Gravel (14) Sand (26)	Gravel (14) Sand (30)
Sizes of filtering materials (mm.)	Eff. size 0.40–0.50 Uniformity coef. 1.65	Eff. size, 0.40 to 0.50	Eff. size 0.47 Uniformity coef. 1.3	Eff. size 0.35–0.44 Uniformity coef. 1.65
Cleaning of filter system	Reverse flow of water. Vertical rise, 27 in. per min.	High velocity wash water	Reverse flow of water. Velocity 2 ft. per minute	Reverse flow of water. Velocity 19 in. per min.
Number of subsiding or coagulating basins	10	2	2†	4–6
Control of rate of filtration	Builder's Iron Foundry	Builder's Iron Foundry Venturi type	Telescopic tube and float maintain constant head on orifice	Simplex rate controller on 16. Builder's Iron Foundry type on 8
Wash water used, per cent. of filtered water	3.0	Expected less than 2 per cent.	2.75 per cent.	2.2 per cent.

* Used only with low alkalinities after storms.
† Capacities: 1.8 mg.; 1.4 mg.
‡ Also settling basin of 100,000,000 gal. capacity.
‖ (1922).
¶ Includes $135,000, for covering and improving filtered water basin.
** Also settling reservoir, capacity 75,000,000.

Table 272. Data Relating to Typical Rapid Filter Plants.—(*Continued*)

	Little Falls, N. J.	Baltimore, Md.	Cambridge	Cincinnati
Plant put in operation	1902*	1915	1923	1907
Designer	George W. Fuller, and others	J. W. Armstrong	G. A. Johnson	G. H. Benzenberg
Population (1920)	Nearly 500,000	632,000	109,456	401,247
Total cost	Including chemical treatment plant, buildings, etc., about $850,000†	$1,472,350 Including pumping station, roads, engineering, legal, real estate ($72,000), and 15 m.g. covered reservoir	$723,900 (contract)	Coagulating basins........ 304,913 Filters........ 734,102‡ Filtered water basin........ 121,362 Pipe lines........ 29,701 Cleaning, grading, etc........ 33,360 Total........ $1,223,438
Cost per million gal. capacity	$20,000	$11,502.81	Undetermined	$10,900
Source of supply	Passaic River	Gunpowder River	Stony and Hobbs Brooks Fresh Pond	Ohio River
Rated capacity (gal. per day)	42,000,000	128,000,000	14,000,000	112,000,000
Total capacity of subsiding or coagulating basins (gal.)	7,000,000	16,000,000	1,500,000	22,973,000
Total capacity of filtered water basin (gal.)	3,500,000	17,500,000	4,000,000	19,000,000
Chemicals used	Sulfate of alumina, soda ash occasionally, chlorine	Alum, lime, liquid chlorine	Sulfate of alumina, soda ash, chlorine	Lime, sulfate of iron
Manner of application of chemicals	Added to coagulant basin through manifold at mouth of main inlet pipe; chlorine to effluent	Alum added at entrance of mixing basin. Lime and chlorine added to filtered water	Vacuum system	Sulfate of iron introduced in "Circulating Chamber;" lime solution on way to coagulation basin
Number of filter units	42	32	10	28
Net area of filter surface (sq. ft.)	15,000	40,320	4,800	39,200
Depths of filtering material (in.)	Gravel (14) Sand (30)	24	36	Gravel (14) Sand (30)
Sizes of filtering materials (mm.)	Eff. size 0.44–0.46. Uniformity coef. 1.50	Varies	0.45	Eff. size 0.39. Uniformity coef. 1.35
Cleaning of filter system	Reverse flow of water with agitation by compressed air for 32 filter.†	Wash water 2 ft. vertical rise per min.	Wash water tank 20,000 gal. per bed	Reverse flow of water. Velocity 17 in. per min.
Number of subsiding or coagulating basins	2	2	....	3
Control of rate of filtration	Simplex Venturi type	Venturi type controllers	....	Balanced valve and orifice controlled by difference in pressure on opposite sides of orifice
Wash water used, per cent. of filtered water	....	1.1 to 2	....	....

* Additions made in 1914; basins designed for 64 mg.
† 10-Filter units with Wheeler bottoms added 1918 and 1919; high-velocity wash.
‡ Not including engineering, legal, or other overhead charges, pipe line to reservoirs or real estate.

Table 272. Data Relating to Typical Rapid Filter Plants.—(*Continued*)

	Cleveland	Columbus, Ohio	Belfast, Me.	Sacramento, Cal.
Plant put in operation	1918	1908	1914	1924
Designer	F. H. Stephenson	J. H. Gregory	R. S. Weston	C. G. Hyde
Population (1920)	925,283	237,031	5,000	68,000
Total cost	$2,225,000	Land ... $ 48,410 Miscellaneous ... 76,490 Pumping station (low lift and direct service) ... 399,240 Purification plant ... 532,480 Engineering ... 81,920 Total ... $1,138,540*	$16,000 including changes in pumping station	Total cost $960,000 including Head house and filters ... $245,000 Land and improvement ... 78,000 Coag. and sed. basins ... 270,000
Cost per million gal. capacity	$14,833.00	$17,750 (purification plant only)	$16,000	$19,300 (weighted)‡
Source of supply	Lake Erie	Scioto River	Little River	Sacramento River
Rated capacity (gal. per day)	150,000,000	30 + 24* = 54 m.g.	1,000,000	32,000,000 to 55,000,000
Total capacity of subsiding or coagulating basins (gal.)	21,000,000	15,000,000	84,000	10,800,000
Total capacity of filtered water basin (gal.)	20,000,000	10 + 6.4† = 16.4 m.g.	31,000	5,000,000
Chemicals used	Sulfate of iron and lime; or sulfate of alumina	Sulfate of alumina, lime, soda ash, chlorine gas, chemicals added to water before entrance to mixing tanks	Sulfate of alumina. Soda ash when necessary. Chemicals added near entrance to coagulating basin	Sulfate of alumina as alum sirup
Manner of application of chemicals	Solutions charged into raw water line through ejectors	...	...	Alum sirup discharged to coagulation plant by blow cases automatically controlled by special air device. Diluted sirup solutions passed through orifices to distribute uniformly
Number of filter units	36	10 + 8† = 18	2	8
Net area of filter surface (sq. ft.)	1454 (each unit)	10,889 + 8,711† = 19,600	384	11,200
Depths of filtering materials (in.)	Gravel (22) Sand (27)	Gravel (10) Sand (30)	Gravel (6) Sand (30)	Gravel (12) and (18)§ Sand (34)
Sizes of filtering material (mm.)	Eff. size, 0.46 Uniformity coef., 1.52	Effect. size 0.41. Uniformity coef., 1.36	Eff. size, 0.41. Uniformity coef. 1.58	Eff. size, 0.40. Uniformity coef., 1.28
Cleaning of filter system	"High" velocity 26 to 29 in. per minute	Reverse flow of water	Reverse flow of water. Velocity = 18 in. per min.	High-velocity wash, 0 to 36 in. per min.
Number of subsiding or coagulating basins	6 (5 in use now)	6	1	8
Control of rate of filtration	Fixed rate; each bed independent	Controller of closed type self-contained and requiring no auxiliary devices	Rate controller of Venturi type	Orifice boxes; float controls
Wash water used, per cent. of filtered water	2	1.7	...	...

* Includes engineering charge on small amount of other work costing $76,490.
† Addition made in 1922.
‡ Excluding special items, $17,500.
§ 12″ with slat bottoms; 18″ with Harrisburg underdrains.

Clark's Experiments with Slow Filters.—Clark[10] has designed and experimented with slow filters which have been charged with aluminum hydrate (or with ferric hydrate) by dosing the sand layers containing magnesium carbonate with aluminum or ferric sulfate in solution. The sand grains become coated with the hydrate precipitated by the magnesium carbonate. The experimental filters remove color effectively and without material increase in hardness or acidity (CO_2). Filters are regenerated by treatment while out of service with caustic soda solution.

Bibliography, Chapter 33. Filtration

1. Gregory: *E. N.*, Oct. 11, 1900, p. 252. **2.** Gregory: *E. R.*, June 29, 1903, p. 656. **3.** Hazen: "American Civil Engineers Hand Book," 4th ed. (Wiley, 1920). **a**, p. 1216; **b**, p. 1215; **c**, p. 1217; **d**, p. 1218; **e**, p. 1220. **4.** Hazen: "Filtration of Public Water Supplies," (Wiley, 1900), p. 37. **5.** *T. A. S. C. E.*, Vol. 57, 1906, p. 327. **6.** *E. N. R.*, Jan. 27, 1921, p. 162. **7.** *E. N. R.*, Apr. 28, 1921, p. 735. **8.** *E. N. R.*, May 17, 1923, p. 882. **9.** *E. N. R.*, Vol. 92, 1924, p. 516, 563. **10.** H. W. Clark: Reports, Mass. State Dept. of Health, 1922, 1923, 1924.

Table 272A. Discharge and Loss of Head, Coffin Rectangular Sluice Gates*

Size of gate, width, height	Area, square feet	Loss of head in feet																	
		0.1	0.25	0.50	0.75	1	2	3	4	5	6	8	10	15	20	25	30	35	40
		Discharge in gallons per minute																	
In.																			
8×8	0.448	341	537	763	933	1,081	1,528	1,867	2,158	2,412	2,640	3,050	3,414	4,165	4,820	5,370	5,900	6,380	6,820
10×10	0.694	529	835	1,180	1,448	1,670	2,365	2,890	3,340	3,735	4,090	4,720	5,288	6,470	7,470	8,350	9,140	9,870	10,270
12×12	1.000	762	1,208	1,709	2,090	2,413	3,430	4,180	4,830	5,400	5,910	6,830	7,622	9,315	10,780	12,030	13,210	14,280	15,270
12×16	1.333	1,016	1,630	2,270	2,780	3,216	4,530	5,540	6,390	7,150	7,810	9,030	10,160	12,420	14,350	16,300	17,560	19,000	20,250
12×20	1.666	1,270	2,080	2,840	3,480	4,020	5,690	6,960	8,040	8,990	9,840	11,350	12,697	15,520	17,950	20,800	22,000	23,780	25,400
16×16	1.777	1,354	2,133	3,025	3,700	4,288	6,050	7,420	8,570	9,570	10,480	12,100	13,544	16,500	19,020	21,330	23,400	25,220	27,000
16×20	2.222	1,694	2,670	3,780	4,640	5,362	7,575	9,275	10,700	11,970	13,100	15,130	16,936	20,700	23,900	26,700	29,250	31,600	33,700
16×24	2.666	2,032	3,230	4,550	5,575	6,433	9,120	11,150	12,880	14,400	15,750	18,200	20,320	24,900	28,720	32,300	35,250	38,000	40,700
18×18	2.250	1,715	2,700	3,825	4,680	5,429	7,670	9,380	10,820	12,100	13,250	15,300	17,149	20,900	24,120	27,000	29,600	31,900	34,200
18×24	3.000	2,287	3,625	5,120	6,270	7,239	10,230	12,520	14,440	16,180	17,700	20,480	22,868	27,950	32,300	36,110	39,500	42,700	45,600
18×36	4.500	3,430	5,400	7,640	9,350	10,859	15,280	18,700	21,600	24,100	26,400	30,650	34,300	41,750	48.300	54,000	59,000	63,800	68,100
20×20	2.777	2,117	3,340	4,730	5,800	6,701	9,450	11,590	13,380	14,920	16,330	18,900	21,167	25,850	29.900	33,400	36,600	39,500	42,300
20×24	3.333	2,541	4,010	5,675	6,960	8,043	11,380	13,920	16,040	17,970	19,670	22,700	25,405	31,000	35,750	40,100	43,900	47,500	50,800
20×36	5.000	3,811	6,050	8,550	10,480	12,066	17,090	20,900	24,130	26,950	29,550	34,200	38,113	46,750	54,000	60,500	66,200	71,500	76,500
24×24	4.000	3,049	4,800	6,800	8,330	9,652	13,600	16,680	19,250	21,500	23,580	27,200	30,490	37,200	42,900	48,000	52,600	56,850	60,800
24×36	6.000	4,574	7,220	10,180	12,480	14,479	20,350	24,950	28,870	32,300	35,350	40,800	45,736	55,800	64,400	72,100	79,000	85,200	91,000
24×48	8.000	6,098	9,620	13,600	16,500	19,305	27,250	33,400	38,600	43,200	47,300	54,600	60,981	74,400	86,000	96,200	105.200	114,000	121,800
24×60	10.000	7,623	12,080	17,090	20,900	24,131	34,250	41,800	48,300	54,000	59,200	68,300	76,227	93,300	107,800	120,800	132,200	142,800	152,800
24×72	12.000	9,147	14,425	20,500	25,000	28,957	40,900	50,100	57,800	64,600	70,800	81,600	91,472	111,800	129,000	144,250	158,000	170,800	182,800
24×96	16.000	12,196	19,200	27,180	33,250	38,610	54,400	66,700	77,100	86,200	94,400	109,000	121,963	148,700	171,900	192,000	210,000	226,800	242,800
24×120	20.000	15,245	24,150	34,200	41,800	48,262	68,300	83,800	96,800	108,000	118,200	136,800	152,454	186,800	216,000	241,500	264,000	285.300	305,000
30×30	6.250	4,764	7,520	10,630	13,020	15,081	21,280	26,050	30,100	33,700	36,800	42,500	47,640	58,100	67,200	75.200	82,200	89,000	95,000
30×36	7.500	5,717	9,000	12,730	15,620	18,098	25,500	31,250	36,200	40,400	44,200	51,000	57,169	69,800	80.800	90,000	98,600	106,500	114,000
30×48	10.000	7,623	12,080	17,090	20,900	24,131	34,200	41,300	48,300	54,000	59,200	68,300	76,227	93,300	187,800	120.800	132,100	142,900	152,800
30×60	12.500	9,528	15,200	21,300	26,100	30,165	42,600	52,200	60,300	67,300	73,800	85,200	95,283	116,800	134,700	150,800	165,000	178,000	190,300
30×120	25.000	19,057	30,100	42,500	52,200	60,328	85,200	102,450	120,800	134,800	147,500	170,300	190,568	232,800	268,800	301,000	329,000	356,000	380,000
36×36	9.000	6,860	10,800	15,250	18,750	21,718	30,600	37,500	43,300	48,400	53,000	61,300	68,604	83,800	96,700	108,000	118,500	128,000	136,800
36×42	10.500	8,004	12,620	17,880	21,880	25,337	35,800	43,800	50,600	56,500	62,000	71,700	80,037	98,000	113,000	126,300	138,300	149,500	159,000
36×48	12.000	9,147	14,425	20,450	25,000	28,957	40,900	50,100	57,800	64,600	70,800	81,600	91,472	111,800	129,000	144,250	158,000	170,800	182,000
36×60	15.000	11,434	18,030	25,520	31,300	36,197	51,200	62,800	72,500	81,000	88,700	102,500	114,341	139,800	161,500	180,300	197,800	213,500	228,000
36×96	24.000	18,293	28,800	40,800	50,000	57,915	81,800	102,000	116,000	129,800	141,800	164,000	182,925	223,500	258,000	288,000	317,000	342,000	365,000
40×40	11.111	8,469	13,500	18,900	23,200	26,812	37,900	46,400	53,700	60,000	65,700	76,000	84,694	103,300	119,000	133,500	146,200	158,000	169,000
40×60	16.666	12,704	20,100	28,400	34,800	40,216	56,800	69,600	80,400	89,900	98,400	113,500	127,038	155,800	179,800	201,000	220,000	237,900	254,000
42×42	12.250	9,338	14,730	20,880	25,600	29,560	41,800	51,300	59,200	66,200	72,400	83,800	93,376	114,200	132,000	147,300	161,000	174,000	186,000
42×60	17.500	13,340	21,000	29,720	36,500	42,229	59,500	72,900	84,300	94,200	103,200	119,200	133,396	162,800	188,000	210,000			

* Free discharge, not submerged, gates wide open.

PART VI
HYDRAULICS AND MATERIALS

CHAPTER 34

HYDRAULIC COMPUTATIONS*

(In following formulas use feet and seconds, unless otherwise noted.)

Precision. For many hydraulic computations in practical waterworks problems, four-place logarithms, slide rules,† and short-cut approximations are quite sufficient for the precision of the data and the necessities of the results. Use of many significant figures is a time-wasting absurdity, *e.g.*, giving total consumption of water, capacity of a reservoir, total pumpage, flow of a stream, and similar quantities to the gallon when millions of gallons or tens of thousands would be appropriate, since inherent uncertainties commonly range from 5 to 25 per cent. Likewise most estimates of total cost should end with two to four ciphers before the decimal point.

Properties of Circular Pipes. (For contents of cylinders see p. 820.) *Hydraulic radius*, **R**:

$$R = A \div P, \text{ where } P = \text{wetted perimeter; } A = \text{area.}$$

For circular cross-section, full or half full,

$$R = \tfrac{1}{4}D \quad (D = \text{diam.})$$

Table 275, p. 752, gives hydraulic radii of pipes, $\frac{1}{2}$ to 144 in. diam.

* "Hydraulic Tables," by Hazen and Williams (John Wiley and Sons, Inc., and "Handbook of Hydraulics," by King (McGraw-Hill Book Company, Inc.), will be found most useful. Other recent books on Hydraulics include:

A. H. Gibson, "Hydraulics and its Applications," Van Nostrand, 1921.
A. A. Barnes, "Hydraulic Flow Reviewed," Spon & Chamberlain, 1916.
E. S. Bellasis, "Hydraulics with Working Tables," Dutton, 1922.
R. L. Daugherty, "Hydraulics," McGraw-Hill Book Company, Inc., 1919.
W. F. Durand, "Hydraulics of Pipe Lines," Van Nostrand, 1921.
H. W. King & C. O. Wisler, "Hydraulics," Wiley, 1922.
F. C. Lea, "Hydraulics," Arnold and Co., 1923.
F. W. Medaugh, "Elementary Hydraulics," Van Nostrand, 1924.
E. H. Lewitt, "Hydraulics," Pitman, 1923.
E. H. Sprague, "Hydraulics," Van Nostrand, 1924.
"Flow of Water in Pipes," by Hiram F. Mills, Am. Academy of Arts & Sciences, Boston, 1925.

† Hydraulic slide rule, devised by Williams and Hazen, Abbott-McKay Corp., Boston is convenient for pipe computations and many other problems.

Table 273. Relative Discharging Capacities of Pipes Flowing Full

Diam., in.	Diam., in.															
	3	4	5	6	8	10	12	14	16	18	20	22	24	30	36	48
48	.....	.....	.....	.....	.....	.....	.....	.....	15.59	11.61	8.92	7.03	5.65	3.24	2.05	1
44	.....	.....	.....	.....	.....	.....	.....	17.50	12.54	9.34	7.17	5.66	4.55	2.61	1.65	..
40	.....	.....	.....	.....	.....	.....	20.23	13.47	9.85	7.34	5.64	4.44	3.57	2.05	1.26	..
36	.....	.....	.....	.....	.....	.....	15.58	8.41	7.59	5.65	4.34	3.42	2.74	1.58	1.	..
33	.....	.....	.....	.....	34.55	19.78	12.54	8.52	6.11	4.55	3.49	2.75	2.21	1.27	.....	..
30	.....	.....	.....	.....	27.09	15.54	9.85	6.54	4.80	3.57	2.74	2.16	1.74	1.	.....	..
27	.....	.....	.....	42.95	16.61	9.96	7.59	5.16	3.70	2.75	2.11	1.67	1.34	.....	.....	..
24	.....	.....	50.50	32.00	15.58	8.92	5.65	3.84	2.75	2.05	1.57	1.24	1.	.....	.....	..
22	.....	70.96	40.65	25.73	12.53	7.17	4.55	3.09	2.16	1.65	1.26	1.	.....	.....	.....	..
20	.....	55.96	32.05	20.29	9.88	5.66	3.58	2.43	1.74	1.30	1.	.....	.....	.....	.....	..
18	.....	42.01	24.63	15.58	7.25	4.34	2.75	1.87	1.34	1.	.....	.....	.....	.....	.....	..
16	65.77	32.01	18.31	11.60	5.65	3.23	2.05	1.39	1.	.....	.....	.....	.....	.....	.....	..
14	47.14	22.94	13.15	8.32	4.05	2.32	1.47	1.	.....	.....	.....	.....	.....	.....	.....	..
12	32.05	15.60	8.93	5.65	2.75	1.57	1.	.....	.....	.....	.....	.....	.....	.....	.....	..
10	20.31	9.88	5.66	3.58	1.74	1.	.....	.....	.....	.....	.....	.....	.....	.....	.....	..
8	11.63	5.66	3.24	2.05	1.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	..
6	5.66	2.75	1.58	1.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	..
5	3.58	1.75	1.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	..
4	2.05	1.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	..
3	1.	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	..

The figures show how many pipes of the sizes printed at the top are equivalent to one pipe of the size in the first column; friction losses taken into account; $Q_1 : Q_2 :: \sqrt{d_1^5} : \sqrt{d_2^5}$.

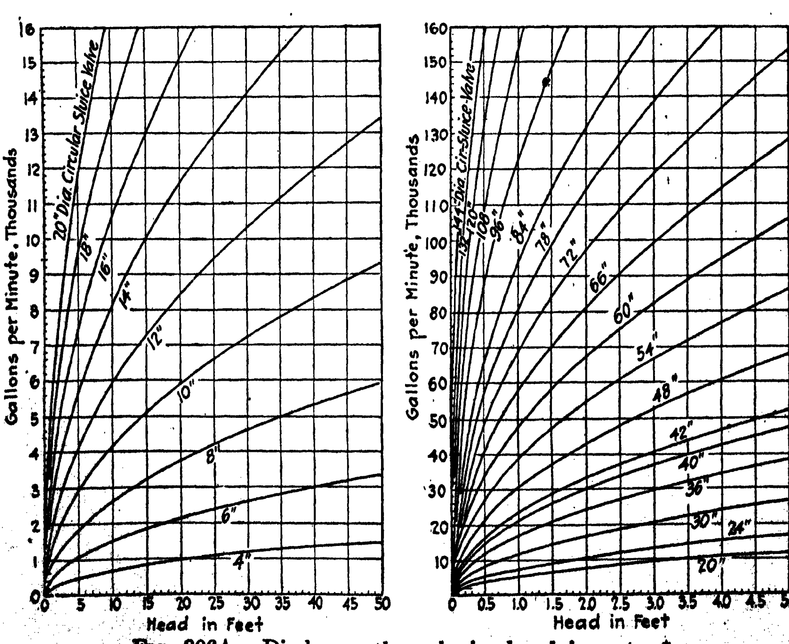


Fig. 306A.—Discharges through circular sluice gates.*

* Wide open, free discharge into air.

Table 274.* **Hydraulic Elements of a Circular Conduit, Partly Filled**
Diameter = 1
John H. Gregory

Depth of filled segment	Area a	Wetted perimeter p	Hydraulic radius r	$\sqrt{r}$	$a\sqrt{r}$	Depth of filled segment
0.01	0.0013293	0.20033	0.0066356	0.081459	0.00010829	0.01
0.02	0.0037485	0.28379	0.013209	0.11493	0.00043081	0.02
0.03	0.0068655	0.34817	0.019719	0.14042	0.00096408	0.03
0.04	0.010538	0.40272	0.026167	0.16176	0.0017046	0.04
0.05	0.014681	0.45103	0.032551	0.18042	0.0026488	0.05
0.06	0.019239	0.49493	0.038872	0.19716	0.0037932	0.06
0.07	0.024168	0.53553	0.045130	0.21244	0.0051343	0.07
0.08	0.029435	0.57351	0.051324	0.22655	0.0066685	0.08
0.09	0.035012	0.60939	0.057454	0.23970	0.0083922	0.09
0.10	0.040875	0.64350	0.063520	0.25203	0.010302	0.10
0.11	0.047006	0.67613	0.069521	0.26367	0.012394	0.11
0.12	0.053385	0.70748	0.075458	0.27470	0.014665	0.12
0.13	0.059999	0.73773	0.081330	0.28518	0.017111	0.13
0.14	0.066833	0.76699	0.087137	0.29519	0.019728	0.14
0.15	0.073875	0.79540	0.092878	0.30476	0.022514	0.15
0.16	0.081112	0.82303	0.098553	0.31393	0.025464	0.16
0.17	0.088536	0.84998	0.10416	0.32274	0.028574	0.17
0.18	0.096134	0.87630	0.10971	0.33122	0.031841	0.18
0.19	0.10390	0.90205	0.11518	0.33938	0.035262	0.19
0.20	0.11182	0.92730	0.12059	0.34726	0.038832	0.20
0.21	0.11990	0.95207	0.12593	0.35487	0.042548	0.21
0.22	0.12811	0.97641	0.13121	0.36223	0.046406	0.22
0.23	0.13646	1.0004	0.13642	0.36935	0.050403	0.23
0.24	0.14494	1.0239	0.14156	0.37624	0.054534	0.24
0.25	0.15355	1.0472	0.14663	0.38292	0.058796	0.25
0.26	0.16226	1.0701	0.15163	0.38939	0.063184	0.26
0.27	0.17109	1.0928	0.15656	0.39568	0.067696	0.27
0.28	0.18002	1.1152	0.16142	0.40178	0.072328	0.28
0.29	0.18905	1.1374	0.16622	0.40770	0.077074	0.29
0.30	0.19817	1.1593	0.17094	0.41345	0.081933	0.30
0.31	0.20738	1.1810	0.17559	0.41904	0.086899	0.31
0.32	0.21667	1.2025	0.18018	0.42447	0.091969	0.32
0.33	0.22603	1.2239	0.18469	0.42975	0.097138	0.33
⅓	0.22917	1.2310	0.18617	0.43148	0.098883	⅓
0.34	0.23547	1.2451	0.18912	0.43489	0.10240	0.34
0.35	0.24498	1.2661	0.19349	0.43988	0.10776	0.35
0.36	0.25455	1.2870	0.19779	0.44473	0.11321	0.36
0.37	0.26418	1.3078	0.20201	0.44945	0.11874	0.37
0.38	0.27386	1.3284	0.20615	0.45404	0.12434	0.38
0.39	0.28359	1.3490	0.21023	0.45851	0.13003	0.39
0.40	0.29337	1.3694	0.21423	0.46285	0.13578	0.40

* See also Fig. 308, p. 753.

Table 274. Hydraulic Elements of a Circular Conduit, Partly Filled.—(*Continued*)
Diameter = 1
John H. Gregory

Depth of filled segment	Area a	Wetted perimeter p	Hydraulic radius r	$\sqrt{r}$	$a\sqrt{r}$	Depth of filled segment
0.41	0.30319	1.3898	0.21815	0.46707	0.14161	0.41
0.42	0.31304	1.4101	0.22200	0.47117	0.14749	0.42
0.43	0.32293	1.4303	0.22577	0.47515	0.15344	0.43
0.44	0.33284	1.4505	0.22947	0.47903	0.15944	0.44
0.45	0.34278	1.4706	0.23309	0.48279	0.16549	0.45
0.46	0.35274	1.4907	0.23663	0.48644	0.17159	0.46
0.47	0.36272	1.5108	0.24009	0.48999	0.17773	0.47
0.48	0.37270	1.5308	0.24347	0.49343	0.18390	0.48
0.49	0.38270	1.5508	0.24678	0.49677	0.19011	0.49
0.50	0.39270	1.5708	0.25000	0.50000	0.19635	0.50
0.51	0.40270	1.5908	0.25314	0.50313	0.20261	0.51
0.52	0.41269	1.6108	0.25620	0.50617	0.20889	0.52
0.53	0.42268	1.6308	0.25918	0.50910	0.21519	0.53
0.54	0.43266	1.6509	0.26208	0.51193	0.22149	0.54
0.55	0.44262	1.6710	0.26489	0.51467	0.22780	0.55
0.56	0.45255	1.6911	0.26761	0.51731	0.23411	0.56
0.57	0.46247	1.7113	0.27025	0.51986	0.24042	0.57
0.58	0.47236	1.7315	0.27280	0.52231	0.24671	0.58
0.59	0.48221	1.7518	0.27527	0.52466	0.25300	0.59
0.60	0.49203	1.7722	0.27764	0.52692	0.25926	0.60
0.61	0.50181	1.7926	0.27993	0.52908	0.26550	0.61
0.62	0.51154	1.8132	0.28212	0.53116	0.27170	0.62
0.63	0.52122	1.8338	0.28423	0.53313	0.27788	0.63
0.64	0.53085	1.8546	0.28623	0.53501	0.28401	0.64
0.65	0.54042	1.8755	0.28815	0.53679	0.29009	0.65
0.66	0.54992	1.8965	0.28996	0.53848	0.29613	0.66
$\frac{2}{3}$	0.55623	1.9106	0.29112	0.53956	0.30011	$\frac{2}{3}$
0.67	0.55936	1.9177	0.29168	0.54008	0.30210	0.67
0.68	0.56873	1.9391	0.29330	0.54157	0.30801	0.68
0.69	0.57802	1.9606	0.29482	0.54297	0.31385	0.69
0.70	0.58723	1.9823	0.29623	0.54427	0.31961	0.70
0.71	0.59635	2.0042	0.29754	0.54548	0.32529	0.71
0.72	0.60538	2.0264	0.29875	0.54658	0.33089	0.72
0.73	0.61431	2.0488	0.29984	0.54758	0.33638	0.73
0.74	0.62313	2.0715	0.30082	0.54847	0.34177	0.74
0.75	0.63185	2.0944	0.30169	0.54926	0.34705	0.75
0.76	0.64045	2.1176	0.30244	0.54994	0.35221	0.76
0.77	0.64893	2.1412	0.30307	0.55051	0.35725	0.77
0.78	0.65728	2.1652	0.30357	0.55097	0.36215	0.78
0.79	0.66550	2.1895	0.30395	0.55131	0.36690	0.79
0.80	0.67357	2.2143	0.30419	0.55154	0.37150	0.80

Table 274. Hydraulic Elements of a Circular Conduit, Partly Filled.—(*Concluded*)
Diameter = 1
John H. Gregory

Depth of filled segment	Area a	Wetted perimeter p	Hydraulic radius r	$\sqrt{r}$	$a\sqrt{r}$	Depth of filled segment
0.81	0.68150	2.2395	0.30430	0.55164	0.37594	0.81
0.82	0.68926	2.2653	0.30427	0.55161	0.38020	0.82
0.83	0.69686	2.2916	0.30409	0.55145	0.38428	0.83
0.84	0.70429	2.3186	0.30376	0.55114	0.38816	0.84
0.85	0.71152	2.3462	0.30327	0.55070	0.39183	0.85
0.86	0.71856	2.3746	0.30260	0.55010	0.39528	0.86
0.87	0.72540	2.4039	0.30176	0.54933	0.39848	0.87
0.88	0.73201	2.4341	0.30073	0.54839	0.40143	0.88
0.89	0.73839	2.4655	0.29949	0.54726	0.40409	0.89
0.90	0.74452	2.4981	0.29804	0.54593	0.40646	0.90
0.91	0.75039	2.5322	0.29634	0.54437	0.40849	0.91
0.92	0.75596	2.5681	0.29437	0.54256	0.41015	0.92
0.93	0.76123	2.6061	0.29210	0.54046	0.41142	0.93
0.94	0.76616	2.6467	0.28948	0.53803	0.41222	0.94
0.95	0.77072	2.6906	0.28645	0.53521	0.41250	0.95
0.96	0.77486	2.7389	0.28291	0.53189	0.41214	0.96
0.97	0.77853	2.7934	0.27870	0.52792	0.41100	0.97
0.98	0.78165	2.8578	0.27351	0.52299	0.40879	0.98
0.99	0.78407	2.9413	0.26658	0.51631	0.40482	0.99
1.00	0.78540	3.1416	0.25000	0.50000	0.39270	1.00

Conduit Partly Filled. Elements are given in Table 274. Area of segment for any circle is found by multiplying diameter squared by coefficient found in "Area of segment" column, opposite "depth of filled segment." For wetted perimeter and hydraulic radius, multiply coefficient, in respective columns, by diameter. Units are homogeneous with those of diameter.

Bernoulli's Theorem for Pipes, Including Losses. Comparing point 1 upstream from 2,

$$H_T = h_1 + \frac{V_1^2}{2g} + Z_1 = h_2 + \frac{V_2^2}{2g} + Z_2 + f_{1-2}$$

wherein

H_T = static head on datum,
Z = height of center of pipe above datum,
V = velocity,
h = piezometric height, *i.e.*, position of hydraulic gradient,
f_{1-2} = losses between points 1 and 2, and may include those of entrance, fluid friction, enlargements, contractions, valves, bends, or obstructions. If H represents difference in elevation between water surface in reservoir and center line of outlet, and V the velocity at outlet, Bernoulli's equation reduces to $H = f_{1-2} + \frac{V^2}{2g}$.

Table 275. Mathematical Properties of Cylindrical Pipes (Full)

Diam.		Powers of diam., D, ft.					Circumference		Hydraulic radius = R, ft.						Area		Contents per lin. ft.*		
in.	ft., D	$1/D$	D^2	D^5	$D^{\frac{5}{2}}$	$\sqrt{D}$	in.	ft.	R	R^2	$R^{\frac{2}{3}}$	$1/R$	$\sqrt{R}$	$(1/R)^{\frac{4}{3}}$	sq. in.	sq. ft.	cu. ft.	gals.	lbs., water
½	0.0416	23.99	0.0017	0.0000001	0.00035	0.204	1.571	0.131	0.0104	0.0001	0.047	96.00	0.102	439.6	0.196	0.001	0.0014	0.0102	0.087
⅝	0.0521	19.19	0.0027	0.0000004	0.00062	0.228	1.964	0.164	0.0130	0.0002	0.055	76.80	0.114	326.4	0.307	0.002	0.0021	0.0159	0.131
¾	0.0625	16.00	0.0039	0.000001	0.0010	0.250	2.356	0.196	0.0156	0.0002	0.062	64.00	0.125	256.0	0.442	0.003	0.0031	0.0230	0.194
1	0.0833	12.00	0.0069	0.000004	0.0020	0.289	3.142	0.262	0.0208	0.0004	0.076	48.00	0.144	174.5	0.785	0.005	0.0055	0.0408	0.344
1¼	0.1042	9.60	0.0108	0.000012	0.0035	0.322	3.927	0.327	0.0260	0.0007	0.088	38.40	0.161	129.5	1.227	0.008	0.0085	0.0638	0.531
1½	0.1250	8.30	0.0156	0.00003	0.0055	0.354	4.712	0.393	0.0312	0.0010	0.099	32.00	0.177	101.6	1.767	0.012	0.0123	0.0918	0.766
1¾	0.1458	6.86	0.0213	0.00007	0.0081	0.382	5.500	0.458	0.0365	0.0013	0.110	27.43	0.191	82.7	2.405	0.017	0.0167	0.1249	1.043
2	0.1667	6.00	0.0279	0.00013	0.0113	0.408	6.283	0.524	0.0417	0.0017	0.120	24.00	0.204	69.2	3.142	0.022	0.0218	0.1632	1.36
2½	0.2083	4.8	0.0434	0.00039	0.01981	0.457	7.854	0.655	0.0521	0.0027	0.139	19.20	0.228	51.4	4.909	0.034	0.0341	0.2550	1.59
3	0.2500	4.00	0.0625	0.00098	0.03125	0.500	9.425	0.785	0.0625	0.0039	0.157	16.00	0.250	40.3	7.069	0.049	0.0491	0.3672	3.06
4	0.3333	3.00	0.1111	0.00412	0.06415	0.577	12.566	1.047	0.0833	0.0069	0.190	12.00	0.289	27.5	12.566	0.087	0.0873	0.6528	5.45
5	0.4167	2.40	0.1736	0.0126	0.1121	0.645	15.708	1.309	0.1042	0.0108	0.221	9.600	0.323	20.4	19.635	0.136	0.1364	1.020	8.53
6	0.5000	2.00	0.2500	0.0312	0.1768	0.707	18.850	1.571	0.125	0.0156	0.250	8.000	0.354	16.00	28.274	0.196	0.1963	1.469	12.27
8	0.6667	1.50	0.4444	0.132	0.3629	0.817	25.133	2.094	0.1667	0.0279	0.303	6.000	0.408	10.90	50.265	0.349	0.3491	2.611	21.8
10	0.8333	1.2	0.6944	0.402	0.6339	0.913	31.416	2.618	0.2083	0.0433	0.351	4.800	0.456	8.10	78.540	0.545	0.5454	4.080	34.0
12	1.000	1.0	1.0	1.0	1.00	1.0	37.699	3.142	0.25	0.0625	0.397	4.000	0.500	6.35	113.100	0.785	0.7854	5.875	49.1
14	1.166	0.857	1.3611	2.16	1.4702	1.080	43.982	3.665	0.2917	0.085	0.440	3.442	0.540	5.20	153.94	1.069	1.069	7.997	66.8
16	1.333	0.75	1.77	4.22	2.0528	1.155	50.266	4.189	0.3333	0.111	0.481	3.000	0.577	4.33	201.06	1.396	1.396	10.44	87.3
18	1.50	0.66	2.25	7.594	2.7557	1.224	56.549	4.712	0.3750	0.141	0.520	2.667	0.612	3.70	254.47	1.767	1.768	13.22	110.5
20	1.66	0.60	2.77	12.858	3.5861	1.291	62.832	5.236	0.4167	0.174	0.558	2.400	0.646	3.21	314.16	2.182	2.182	16.32	136.4
24	2.0	0.50	4.0	32.0	5.6569	1.414	75.398	6.283	0.5000	0.250	0.630	2.000	0.707	2.52	452.39	3.142	3.142	23.50	194
30	2.5	0.40	6.25	97.656	9.8821	1.581	94.248	7.854	0.6250	0.391	0.731	1.600	0.790	1.87	706.86	4.909	4.909	36.72	306
36	3.0	0.33	9.0	243.06	15.588	1.732	113.097	9.425	0.7500	0.563	0.826	1.333	0.866	1.47	1017.90	7.069	7.069	52.88	442
40	3.33	0.30	11.11	411.52	20.286	1.825	125.664	10.472	0.8333	0.694	0.885	1.200	0.913	1.28	1256.6	8.727	8.727	65.28	546
42	3.5	0.285	12.25	525.22	22.918	1.871	131.947	10.996	0.8750	0.766	0.915	1.143	0.935	1.19	1385.4	9.621	9.621	71.97	601
48	4.0	0.25	16.0	1024	32.0	2.0	150.796	12.566	1.000	1.000	1.000	1.000	1.000	1.00	1809.6	12.566	12.566	94.00	786
54	4.5	0.222	20.25	1845	42.957	2.121	169.646	14.137	1.125	1.263	1.081	0.888	1.060	0.854	2290.2	15.904	15.90	118.95	994
60	5.0	0.20	25.0	3125	55.902	2.236	188.496	15.708	1.250	1.563	1.161	0.800	1.118	0.74	2827.4	19.635	19.63	146.88	1227
66	5.5	0.182	30.25	5032	70.943	2.345	207.345	17.279	1.375	1.891	1.237	0.727	1.173	0.653	3421.2	23.758	23.76	177.72	1485
72	6.0	0.166	36.0	7776	88.182	2.449	226.195	18.850	1.5	2.250	1.310	0.667	1.225	0.583	4071.5	28.274	28.27	211.51	1767
78	6.5	0.154	42.25	11603	107.72	2.550	245.044	20.420	1.625	2.641	1.382	0.615	1.275	0.523	4778.4	33.183	33.18	248.23	2074
84	7.0	0.143	49.0	16807	129.64	2.646	263.894	21.991	1.75	3.063	1.453	0.571	1.323	0.474	5541.8	38.485	38.48	287.88	2405
90	7.5	0.133	56.25	23730	154.05	2.739	282.743	23.562	1.875	3.516	1.521	0.533	1.369	0.432	6361.7	44.179	44.18	330.48	2761
96	8.0	0.125	64.0	32768	181.02	2.828	301.593	25.133	2.000	4.000	1.587	0.500	1.414	0.397	7238.2	50.265	50.27	376.01	3141
102	8.5	0.118	72.25	44371	210.64	2.915	320.442	26.704	2.125	4.516	1.653	0.4706	1.458	0.366	8171.3	56.745	56.75	424.48	3546
108	9.0	0.111	81.0	59049	243.00	3.000	339.292	28.274	2.25	5.063	1.717	0.4444	1.500	0.339	9160.9	63.617	63.62	475.89	3976
114	9.5	0.105	90.25	77378	278.17	3.082	358.142	29.845	2.375	5.641	1.780	0.4210	1.541	0.316	10207.0	70.882	70.88	530.24	4430
120	10.0	0.10	100.0	100000	316.23	3.162	376.991	31.416	2.500	6.250	1.842	0.4000	1.581	0.295	11309.7	78.540	78.54	587.52	4910
126	10.5	0.095	110.25	127628	357.25	3.240	395.841	32.987	2.625	6.891	1.903	0.3310	1.620	0.276	12469.0	86.590	86.59	647.74	5410
132	11.0	0.091	121.0	161051	401.31	3.317	414.690	34.558	2.75	7.563	1.963	0.3636	1.658	0.259	13684.8	95.033	95.03	710.90	5940
138	11.5	0.087	132.25	201137	448.48	3.391	433.540	36.178	2.875	8.266	2.022	0.3478	1.696	0.244	14957.1	103.869	103.87	776.99	6490
144	12.0	0.083	144.0	248832	498.83	3.464	452.389	37.699	3.000	9.000	2.080	0.3333	1.732	0.231	16286.0	113.097	113.10	846.03	7070

* A pipe 1 yd. long holds approximately as many pounds of water as the square of its diam. in in. (see also p. 820).

Table 276. Theoretical Velocity* Required to Move Loose Particles† of Various Sizes by Jet Action[2]

Diam. of particle, in mm.	Velocity, in ft. per min., required to move by jet action	Diam. of particle, in mm.	Velocity, in ft. per min., required to move by jet action
5.0	43.4	0.1	6.0
3.0	33.7	0.08	5.5
1.0	19.4	0.05	4.3
0.8	17.4	0.03	3.4
0.5	13.8	0.01	1.94
0.3	9.6	....	....

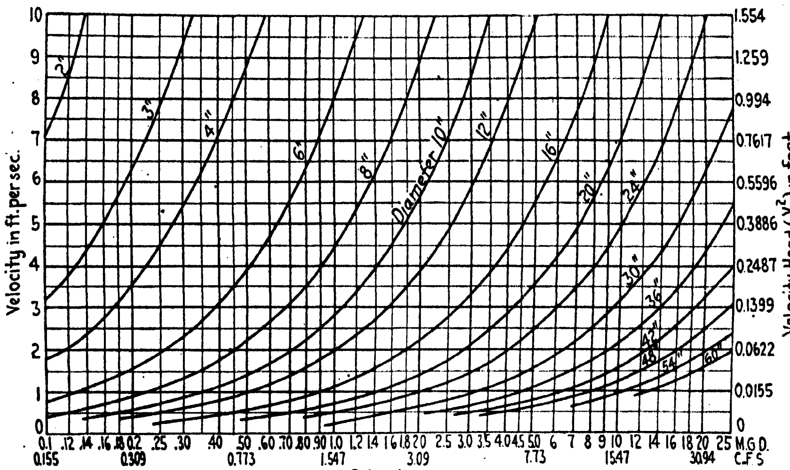


FIG. 307.—Velocity and velocity-head diagram (for circular pipes in terms of cfs. and mgd.).

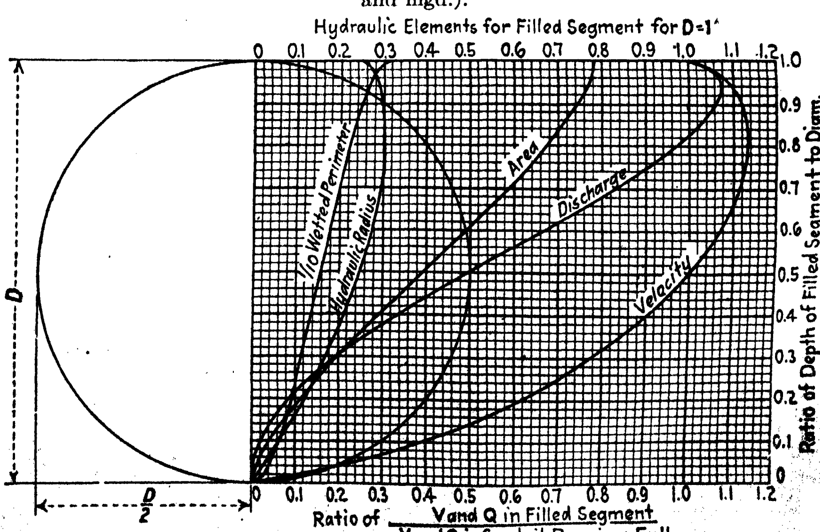


FIG. 308.—Circular conduit. Hydraulic elements. (See also Table 274.)

* See also p. 276.
† Weight assumed at 159.5 lbs. per cu. ft.

FORMULAS FOR FLOW IN PIPES*

"Long" and "Short" Pipes.[75] Be careful not to take from tables or formulas for "long" pipes, discharges, etc., for pipes of insufficient length to be classed as "long." In such cases use formula for "short" pipes: $Q = CA\sqrt{2gh}$. Q = discharge, *cfs*; A = area, sq. ft.; g = acceleration of gravity; h = head on center of pipe, ft. Coefficient C varies with ratio of pipe diam., d, to length, l; if $\frac{l}{d} = 1$, $C = 0.62$; $\frac{l}{d} = 3$, $C = 0.815$; $\frac{l}{d} = 25$, $C = 0.71$; $\frac{l}{d} = 100$, $C = 0.55$. To be "long," a pipe should have length in excess of 500 diam.; if length is less than 50 diam., pipe is "very short."

Chezy Formula. $V = C\sqrt{rs}$; V is the mean velocity, r, hydraulic radius, and s, head lost divided by length on invert. Since C† is a function of v and r, irrespective of surface, this formula fails to express accurately the law of fluid friction.[4a] The error is not great, however, and simplicity of form has led to universal use. Darcy proposed substitution of $\sqrt{\frac{8g}{f}}$ for C; g is acceleration of gravity, and f, coefficient of fluid friction,‡ so that $H_f = \frac{4fLV^2}{D2g}$, in which D is internal diam. of pipe and L, length on slope.

Other exponential formulas may be expressed in the form:

$$H_f = MV^n = \frac{K}{d^p} V^n$$

Williams and Hazen: §	$H_f = \frac{K_W}{d^{1.25}} V^{1.87}.$
Tutton:[5]	$H_f = \frac{K_W}{d^{1.29}} V^{1.96}.$
Manning:[6a]	$H_f = \frac{n^2}{2.208d^{1.33}} V^{2.0}.$
Unwin:[7]	$H_f = \frac{K_V}{d^{1.16 \text{ to } 1.39}} V^{2.0 \text{ to } 1.75}.$
Barnes:[8]	$H_f = \frac{K_B}{d^{0.846 \text{ to } 1.454}} V^{1.692 \text{ to } 2.066}$
Lea:[9]	$H_f = \frac{K_L}{d^{1.25}} V^2.$
Darcy:	$H_f = \frac{K_D}{d} V^2.$
Reclamation Service:[10]	$H_f = \frac{K_R}{d^{1.26}} V^{1.8}.$

* For flow in open channels, see p. 270.
† Values of f and of C are given in "Handbook of Hydraulics," by H. W. King. (McGraw-Hill Book Company, Inc., 1918.)
‡ For M. I. T. investigations for fluids, see *E. N. R.*, Oct. 26, 1922, p. 690.
§ See also p. 765. See chart for converting Kutter's "n" into the Hazen and William's coefficient, *E. N. R.*, June 10, 1924.

Table 277. Loss of Head Due to Friction in 1000 Ft. of Straight Pipe for a Diameter of 1 Ft.*

To Determine the Friction Head Lost per 1000 Ft. of Pipe of Any Other Diameter, Multiply the Values in This Table by $\frac{1}{d^{1.25}}$.

Values of $\frac{1}{d^{1.25}}$ are Given in Table 278

Velocity	Clean cast-iron pipe		Old cast-iron pipe		Clean riveted pipe		Galvanized pipe		Smooth asphalted pipe		Clean wooden pipe		Concrete pipe	
	From	To	From	To	From	To	From	To	From	To	From	To	From	To
0.5	0.09	0.11	0.12	0.17	0.11	0.13	0.10	0.12	0.09	0.11	0.11	0.15	0.12	0.17
1.	0.29	0.42	0.47	0.69	0.40	0.54	0.35	0.45	0.30	0.38	0.37	0.53	0.40	0.68
2.	1.03	1.64	1.82	2.85	1.53	2.29	1.22	1.75	1.01	1.34	1.24	1.86	1.35	2.74
3.	2.11	3.65	4.00	6.50	3.34	5.32	2.52	3.88	2.07	2.78	2.50	3.86	2.74	6.16
4.	3.56	6.44	6.98	11.68	5.82	9.68	4.23	6.81	3.43	4.70	4.13	6.52	4.53	10.94
5.	5.33	10.00	10.76	18.45	8.98	15.40	6.34	10.58	5.09	7.04	6.10	9.76	6.73	17.10
6.	7.4	14.3	15.3	26.8	12.8	22.5	8.8	15.1	7.1	9.8	8.4	13.6	9.2	24.6
7.	9.8	19.4	20.7	36.6	17.2	31.0	11.6	20.5	9.3	12.9	11.0	17.9	12.2	33.5
8.	12.3	25.3	26.8	48.1	22.2	40.9	14.7	26.6	11.6	16.4	13.8	22.8	15.3	43.8
9.	15.4	31.9	33.7	61.2	27.8	52.2	18.3	33.4	14.4	20.3	16.9	28.3	18.8	55.4
10.	18.7	39.2	41.4	75.9	34.1	65.0	22.1	41.1	17.3	24.6	20.2	34.2	22.6	68.4
11.	22.2	47.3	49.8	92.2	41.0	79.2	26.3	49.5	20.4	29.3	24.0	40.7	26.7	82.8
12.	26.0	56.2	58.9	110.1	48.6	95.0	30.7	58.7	23.7	34.3	28.0	47.5	31.1	98.6
13.	30.0	65.7	68.8	129.5	56.7	112.0	35.5	68.6	27.3	39.6	32.3	55.0	35.8	115.5
14.	34.1	76.1	79.3	150.6	65.5	130.8	40.5	79.4	31.3	45.2	36.7	63.0	40.7	134.0
15.	38.5	87.1	90.6	173.4	74.9	151.0	45.8	90.9	35.3	51.1	41.3	71.4	45.8	153.9
16.	43.	99.	103.	198.	85.	173.	52.	103.	40.	57.	46.	80.	51.	175.
17.	48.	112.	115.	224.	95.	196.	58.	116.	44.	64.	51.	90.	57.	198.
18.	54.	125.	128.	252.	106.	221.	64.	130.	49.	71.	57.	99.	63.	222.
19.	59.	139.	143.	282.	118.	247.	70.	145.	54.	78.	62.	110.	70.	247.
20.	65.	154.	159.	312.	130.	275.	77.	160.	58.	86.	68.	120.	76.	274.
21.	71.	169.	175.	345.	143.	304.	84.	176.	64.	94.	74.	131.	83.	302.
22.	77.	185.	191.	379.	156.	335.	92.	193.	69.	102.	80.	143.	90.	331.
23.	83.	202.	208.	415.	170.	367.	99.	210.	75.	111.	87.	155.	97.	362.
24.	89.	220.	226.	452.	185.	401.	107.	229.	81.	120.	93.	167.	105.	394.
25.	95.	240.	245.	492.	200.	437.	115.	248.	87.	129.	100.	180.	113.	428.

* Reproduction of Table 59, p. 174, "Handbook of Hydraulics," by H. W. King (McGraw-Hill Book Company, Inc., 1918).

Table 278. Values of $\frac{1}{D^{1.25}}$ *

For Other Properties of D, See p. 752

Diam. Ins.	Diam. Ft.	$\frac{1}{D^{1.25}}$ (Ft.)	Diam. Ins.	Diam. Ft.	$\frac{1}{D^{1.25}}$ Ft.	Diam. Ins.	Diam. Ft.	$\frac{1}{D^{1.25}}$ Ft.
½	0.042	53.12	30	2.5	0.318	120	10.0	0.0562
1	0.083	22.39	36	3.0	0.2533	126	10.5	0.526
2	0.167	9.390	42	3.5	0.2089	132	11.0	0.499
3	0.25	5.657	48	4.0	0.1768	138	11.5	0.456
4	0.333	3.948	54	4.5	0.1526	144	12.0	0.416
6	0.5	2.378	60	5.0	0.1337	......	......	......
8	0.667	1.660	66	5.5	0.1187	......	......	......
10	0.833	1.256	72	6.0	0.1065	......	......	......
12	1.000	1.000	78	6.5	0.0963	......	......	......
14	1.167	0.825	84	7.0	0.0878	......	......	......
16	1 333	0.698	90	7.5	0.0806	......	......	......
18	1.5	0.602	96	8.0	0.0743	......	......	......
20	1.667	0.528	102	8.5	0.0689	......	......	......
22	1.833	0.469	108	9.0	0.0641	......	......	......
24	2.0	0.420	114	9.5	0.0600	......	......	......

* Abridged from Table 60, p. 175, "Handbook of Hydraulics," by H. W. King (McGraw-Hill Book Company, Inc., 1918).

Effect of Water Temperatures.[12] For $V = 6$ ft., Butcher concludes that Chezy's $C = 108$ at 40°F., 112 at 55°F., and 116 at 70°F.

Flow in capillary tubes (Hazen)[3c]. $V = csd^2\left(\frac{t+10}{60}\right)$; c being a coefficient (average value = 495), s the slope, d the diam. in in. and v the velocity, ft. per sec., and t the temperature in degrees F. The factor $\left(\frac{t+10}{60}\right)$ was deduced from experiments at Lawrence to represent the flow of water, at different temperatures, through capillary tubes. It also represents the relative rates at which water passes through sand, and at which silt settles through quiet water. This term is more convenient for practical use than the conventional factor of Poiseuille and is sufficiently accurate. See also M. I. T. tests on viscous liquids, *E. N. R.*, Oct. 26, 1922, p. 690.

FLOW IN CAST-IRON, STEEL AND WROUGHT-IRON PIPES

Effect of Age.* With increasing age of service, metal pipes commonly become corroded and tuberculated,† which diminishes the discharge under the same head (both from increased roughness and diminished sectional area). E. B. Weston recommends that friction head for a given Q be taken as 16 per cent. greater than when the pipe is new and clean, for each 5 years of age; that is, for an age of 15 years, take as friction head the value obtained by multiplying the result given by the diagram[13a] by 1.48. Hazen and Williams ("Hydraulic Tables") assign coefficients according to age.‡ A leakage survey at Hammond,

* See particularly "Hydraulics" by Hughes and Safford (The Macmillan Company, 1926), p. 220.

† See pp. [illegible], 387, 416, 813.

‡ The designer must realize that conduits are rarely subject to full flow demand until aged; he must therefore base his calculations on tuberculated conditions.

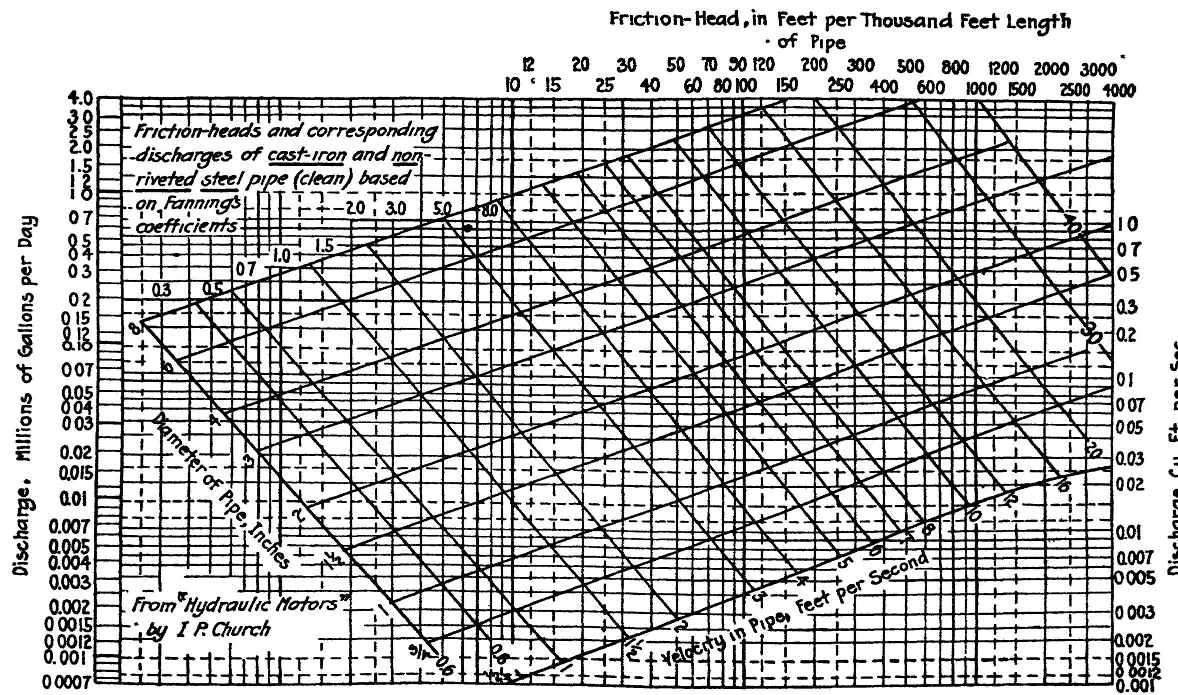


FIG 309.*—Flow in $\frac{1}{2}$ in. to 8 in. clean wrought-iron or cast-iron pipes. (Larger pipes, Fig 310, p. 758.)

(I. P. Church.)31b

* For methods of constructing diagrams involving more than 2 variables, see Harvard Engineering Jl., June, 1912, p. 78. Also, for construction of logarithmic diagram covering the general case, see Eng. Rec., Sept. 3, 1904

Ind., (reported by the Cast-iron Pipe Publicity Bureau), disclosed flow in a 25-year old 24-in. pipe equal to that in a new pipe. (C_H = 134.7, n = 0.0107). See also pp. 421 and 422. Schoder[14] expresses effect of condition of pipe on friction head as follows:

Condition of pipe	H_f	
Very smooth pipes (seamless drawn brass tubes)	$0.30\frac{V^{1.75}}{D^{1.25}}$	$1.99H_f^{0.57}D^{0.71}$
Ordinary cast-iron pipes	$0.38\frac{V^{1.86}}{D^{1.25}}$	$1.69H_f^{0.54}D^{0.67}$
Foul or tuberculated cast-iron pipes	$0.69\frac{V^{2}}{D^{1.25}}$	$1.20H_f^{0.50}D^{0.62}$
Lap-riveted steel pipes, several years old	$0.50\frac{V^{1.95}}{D^{1.25}}$	$1.42H_f^{0.51}D^{0.64}$

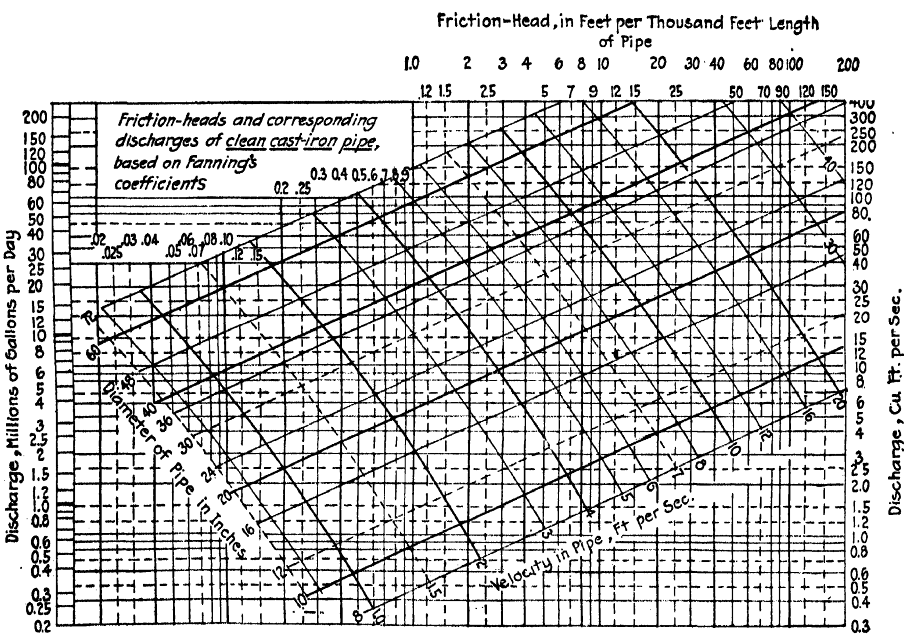


FIG. 310.—Flow in 8 in. to 72 in. clean cast-iron pipes. (Smaller pipes, Fig. 309, p. 757.) (I. P. Church.)[13c]

Probably no wise engineer would attempt to predict the discharge of a 5-year-old cast-iron pipe line within 10 per cent. The above formulas will be on the side of the truth, and within 20 per cent. thereof. At Hartford, 4 and 6-in. pipes, 57 years old, showed values of C_H below 45 in one case, and below 33 in another.[15]

Test Data for Large Cast-iron Mains. Tests on the 30-in. and 36-in. *Fisher Hill force main*, Boston, laid in 1887, 5470 ft. long, with velocities from 2.52 to 2.64 ft. per sec., gave values of Chezy's C from 105 to 117, averaging

Table 279. Flow of Water in New and Old Cast-iron Pipes of Ordinary Sizes in Distribution Systems Quantities, Velocities, and Friction Losses

Diam. in.	Vel. = 3 ft. per sec. Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft.	Vel. = 4 ft. per sec. Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft.	Vel. = 5 ft. per sec. Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft.	Vel. = 6 ft. per sec. Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft.	Vel. = 7 ft. per sec. Mil. gals. per day	Cu. ft. per sec.	Friction head per mile	Friction head per 1000 ft.
4	0.169	0.262	**310.** 55.	**59.** 10.4	0.226	0.34	**553.** 99.	**105.** 187.	0.283	0.43	**863.** 154.	**163.** 29.1	0.34	0.52	**1244.** 227.	**236.** 42.	0.39	0.61	**1660.** 302.	**314.** 57.
6	0.381	0.589	**157.** 34.3	**29.8** 6.5	0.508	0.78	**281.** 61.	**53.** 11.6	0.635	0.98	**438.** 96.	**83.** 18.1	0.76	1.18	**631.** 138.	**120.** 26.1	0.88	1.38	**843.** 188.	**160.** 35.5
8	0.677	1.047	**100.** 24.8	**18.8** 4.70	0.902	1.39	**178.** 44.3	**33.7** 8.4	1.129	1.74	**277.** 69.	**53.** 13.1	1.35	2.10	**399.** 100.	**76.** 18.9	1.58	2.44	**533.** 136.	**101.** 25.7
10	1.058	1.636	**71.** 19.4	**13.4** 3.68	1.410	2.18	**126.** 34.7	**23.9** 6.6	1.76	2.72	**197.** 54.	**37.3** 10.3	2.11	3.27	**284.** 78.	**54.** 14.8	2.46	3.82	**379.** 106.	**72.** 20.1
12	1.523	2.356	**54.** 15.9	**10.2** 3.02	2.031	3.14	**96.** 28.5	**18.3** 5.4	2.53	3.92	**151.** 44.4	**28.5** 8.4	3.05	4.71	**217.** 64.	**41.1** 12.1	3.55	5.5	**289.** 87.	**55.** 16.5
14	2.073	3.207	**43.4** 13.83	**8.2** 2.62	2.763	4.27	**78.** 24.1	**14.7** 4.57	3.45	5.34	**121.** 37.6	**22.9** 7.1	4.15	6.41	**174.** 54.	**33.0** 10.3	4.83	7.48	**233.** 74.	**44.1** 14.0
16	2.707	4.189	**35.9** 11.7	**6.8** 2.22	3.601	5.58	**64.** 20.93	**12.2** 3.96	4.51	6.98	**100.** 32.6	**18.9** 6.2	5.42	8.39	**144.** 47.0	**27.3** 8.9	6.32	9.77	**192.** 64.	**36.4** 12.1
18	3.426	5.30	**30.5** 10.3	**5.8** 1.96	4.569	7.07	**55.** 18.5	**10.3** 3.50	5.71	8.83	**85.** 28.8	**16.1** 5.5	6.85	10.60	**123.** 41.6	**23.2** 7.9	7.99	12.37	**164.** 57.	**31.0** 10.7
20	4.230	6.54	**26.5** 9.24	**5.0** 1.75	5.64	8.73	**47.4** 16.5	**9.0** 3.13	7.05	10.91	**74.** 25.8	**14.0** 4.89	8.46	13.09	**106.** 37.2	**20.2** 7.04	9.87	15.27	**142.** 51.6	**26.9** 9.6
24	6.09	9.42	**20.8** 7.7	**3.95** 1.45	8.12	12.57	**37.3** 13.7	**7.0** 2.59	10.15	15.71	**58.** 21.3	**11.0** 4.04	12.18	18.85	**84.0** 30.7	**15.9** 5.8	14.21	21.99	**112.** 41.9	**21.2** 7.9
30	9.52	14.73	**15.7** 6.1	**2.97** 1.15	12.69	19.63	**28.0** 10.9	**5.3** 2.06	15.86	24.54	**43.7** 16.9	**8.3** 3.21	19.03	29.45	**63.** 24.4	**11.9** 4.62	22.21	34.36	**84.** 33.2	**15.9** 6.39
36	13.71	21.21	**12.5** 5.0	**2.37** 0.95	18.28	28.27	**22.4** 9.0	**4.24** 1.70	22.84	35.34	**34.9** 14.0	**6.6** 2.66	27.41	42.41	**50.** 20.2	**9.5** 3.83	31.98	49.48	**67.** 27.5	**12.7** 5.2
40	16.92	26.181	**11.0** 4.5	**2.09** 0.85	22.56	34.91	**19.7** 8.1	**3.73** 1.53	28.05	43.63	**30.8** 12.6	**5.8** 2.39	33.84	52.4	**44.3** 18.1	**8.4** 3.43	39.48	61.1	**59.** 24.5	**11.2** 4.65
42	18.66	28.86	**10.4** 4.28	**1.97** 0.81	24.87	38.48	**18.6** 7.7	**3.53** 1.45	31.09	48.11	**29.0** 12.0	**5.5** 2.27	37.31	57.7	**41.8** 17.3	**7.9** 3.27	43.53	67.3	**56.** 23.5	**10.6** 4.45
48	24.37	37.70	**8.9** 3.75	**1.68** 0.71	32.49	50.3	**15.9** 6.7	**3.01** 1.27	40.61	62.8	**24.8** 10.4	**4.69** 1.98	48.73	75.4	**35.7** 15.0	**6.8** 2.85	56.8	88.0	**47.6** 20.5	**9.0** 3.88
54	30.84	47.71	**7.8** 3.32	**1.46** 0.63	41.12	63.6	**13.8** 6.00	**2.62** 1.13	51.4	79.5	**21.5** 9.3	**4.08** 1.76	61.7	95.4	**31.0** 13.4	**5.9** 2.53	71.9	111.3	**41.4** 18.2	**7.9** 3.44
60	38.07	58.9	**6.9** 3.01	**1.30** 0.57	50.8	78.5	**12.2** 5.3	**2.32** 1.01	63.5	98.2	**19.1** 8.3	**3.62** 1.58	76.1	117.8	**27.6** 12.0	**5.2** 2.27	88.8	137.4	**36.8** 16.3	**7.0** 3.09
66	46.07	71.3	**6.2** 2.69	**1.17** 0.51	61.4	95.0	**11.0** 4.86	**2.08** 0.92	76.7	118.8	**17.2** 7.5	**3.25** 1.43	92.1	142.5	**24.7** 10.9	**4.68** 2.06	107.5	166.3	**32.9** 14.8	**6.2** 2.81
72	54.8	84.8	**5.6** 2.48	**1.06** 0.47	73.1	113.1	**10.0** 4.44	**1.89** 0.84	91.4	141.4	**15.5** 6.9	**2.94** 1.31	109.6	169.6	**22.4** 10.0	**4.24** 1.89	127.9	197.9	**29.9** 13.6	**5.7** 2.57

The heavier type refers to old pipe. Computed from E. B. Weston's formula: New pipe, $H = \left(0.019892 + \frac{0.00166573}{D}\right)\frac{L}{D}\frac{V^2}{2g}$. **Old pipe,** $H = \left(0.03978 + \frac{0.03332}{D}\right)\frac{L}{D}\frac{V^2}{2g}$. H = loss of head due to friction, ft. D = pipe diameter, ft. L = pipe length, ft.[16]

Table 280.* Flow of Water in New House-service Pipes
(Thompson Meter Company)

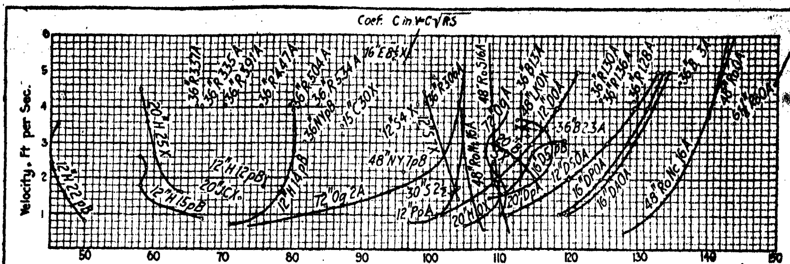
Pressure in main, pounds per square inch	Discharge in cubic feet per minute							
	Nominal internal diameter of pipe (inches)							
	½	¾	1	1½	2	3	4	6
Through 35 feet of service pipe, no back pressure								
30	1.10	3 01	6.13	16.58	33.34	88.16	173.85	444.63
40	1.27	3.48	7.08	19.14	38.50	101.80	200.75	513.42
50	1.42	3.89	7 92	21.40	43.04	113.82	224.44	574.02
60	1 56	4 26	8 67	23 44	47 15	124 68	245.87	628.81
75	1.74	4.77	9.70	26.21	52.71	139 39	274.89	703.03
100	2.01	5.50	11.20	30 27	60.87	160.96	317.41	811.79
130	2 29	6.28	12 77	34 51	69 40	183 52	361.91	925.58
Through 100 feet of service pipe, no back pressure								
30	0.66	1 84	3 78	10.40	21 30	58.19	118.13	317.23
40	0.77	2 12	4.36	12.01	24.59	67.19	136.41	366.30
50	0.86	2 37	4 88	13.43	27.50	75.13	152.51	409.54
60	0 94	2 60	5 31	14 71	30 12	82 30	167.06	448.63
75	1.05	2 91	5 97	16 45	33.68	92.01	186 78	501.58
100	1.22	3.36	6.90	18 99	38 89	106 24	215 68	579.18
130	1 39	3 83	7 86	21 66	44 34	121 14	245 91	660.36
Through 100 feet of service pipes, and 15 feet vertical rise								
30	0.55	1.52	3.11	8.57	17.55	47.90	96.17	260.56
40	0.66	1 81	3.72	10.24	20 95	57.20	116.01	311.09
50	0.75	2 06	4 24	11 67	23.87	65.18	132.20	354.49
60	0.83	2 29	4 70	12 94	26 48	72 28	146 61	393.13
75	0 94	2 59	5.32	14 64	29 96	81.79	165.90	444.85
100	1 10	3 02	6.21	17 10	35.00	95.55	193 82	519.72
130	1 26	3 48	7.14	19 66	40 23	109.82	222 75	597.31
Through 100 feet of service pipe, and 30 feet vertical rise								
30	0.44	1 22	2.50	6.80	14.11	38.63	78 54	211.54
40	0 55	1.53	3.15	8.68	17.79	48.68	98.98	266.59
50	0 65	1.79	3.69	10.16	20.82	56.98	115.87	312.08
60	0.73	2.02	4.15	11.45	23.45	64.22	130.59	351.73
75	0.84	2 32	4 77	13.15	26.95	73.76	149.99	403.98
100	1 00	2.75	5 65	15.58	31.93	87.38	177.67	478.55
130	1.15	3.19	6 55	18.07	37.92	101.33	206.04	554.96

* See also pp. 433 and 434.

111; Kutter's $n = 0.0124$. Values are approximate only, being incidental to engine test. When pipe was 10 years old, tests by C. W. Sherman on a 5154-ft. length gave $C = 103$, and $n = 0.0133$. For a 36-in. main laid in 1894, incidental to an engine test 1 year later, $C = 136.6$ for $V = 4.7$; when the pipe was 2 years old, the following values of coefficients were obtained:

V ft. per sec.	1.1	1.5	2.0	2.2	2.6	3.1	3.5	3.7	4.3	4.5
n.....	0.0116	0.0121	0.0123	0.0125	0.0125	0.0127	0.0127	0.0127	0.0126	0.0127
C.....	117.0	114.9	114.9	113.3	113.7	113.5	113.7	113.7	113.0	114.0

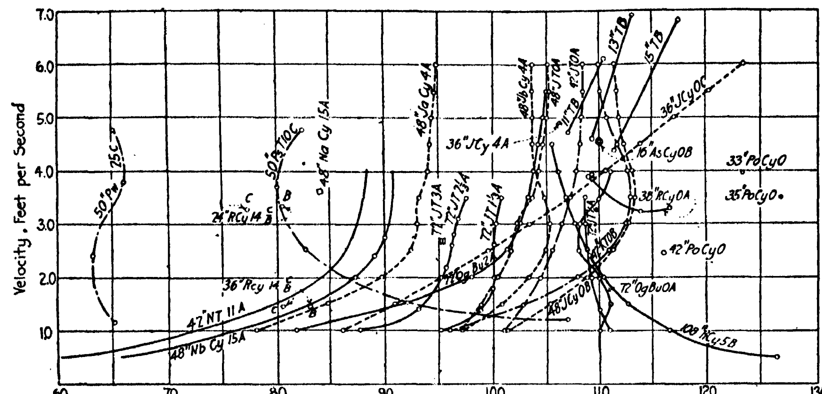
Experiments in 1897, with $V = 3.2$, gave $C = 114$, $n = 0.0126$.



MARK ON DIAGRAM	DIAM.	LOCATION, COATING AND EXPERIMENTER	C	WT. OF EXPT.	AGE YEARS	CONDITION	FIRST USED	SOURCE OF DATA	WEIGHT OF EXPERIMENT
72"Og A	72	Ogden, Utah (Steel all butt joints) Asphalt	108± to 111±	A	New		1897	Tr. A.S.C.E. V.40 p.471	
72"Og 2A	72	" " " " " " "	73.4 " 104.9	"	2	Not Examined	"	" V.44 p.34	
61½"Ro OA	61½	Rosemary, Coal Tar, Fitz Gerald	149.6 " 154.3	"	New		1898±	" p.88	
48"Ro OA	48	" " " Stearns	140.1 " 144.1	"	"		1878	" V.35 p.259	
48"Ro S16A	"	" S. Pipe; Fitz Gerald	108.6 " 111.3	"	16	Tuberculated	"	" "	
48"Ro Nt16A	"	" N. " "	104.4 " 107.8	"	"	"	"	" "	
48"Ro Nc16A	"	" " "	127.7 " 144	"	"	Cleaned	"	" "	
48"NY 7pB	"	N.Y. 10th Ave Uncoated, Duane	96	pB	7	Tuberculated	1873±	" V.28	
48"LKOX	"	Loch Katrine, Asphalt; Jas. M. Gale	112.4	X	New	Good	1869	Hering's Kutter	A=Excellent
36"R1.28A	36	Rochester, Tar, Kuichling	129.5	A	1.28				B=Good
36"R1.30A	"	" " "	125.3	"	1.30				C=Bad
36"R1.36A	"	" " "	125.3	"	1.36				X=Unknown
36"R3.06A	"	" " "	99.2	"	3.06	Heavy			Quality
36"R3.35A	"	" " "	66.8	"	3.35	Growth			p=Probably
36"R3.37A	"	" " "	66.2	"	3.37	of	1894	Tr. A.S.C.E. V.44 p.56	
36"R3.91A	"	" " "	70.2	"	3.91	Sponge			
36"R4.47A	"	" " "	75.3	"	4.47	and			
36"R5.04A	"	" " "	79.8	"	5.04	Polyzoa			
36"R5.34A	"	" " "	81.9	"	5.34				
36"NYpB	"	N.Y. Croton, Kirkwood, 1867	81.6	pB		Heavily Tuberculated		Hering's Kutter	
30"S 2½X	30	Simpson	104	X	2 or 3			Tr. A.S.C.E. V.35 p.278	
20"B 5X	20	Brush	109 to 115	"	5			" "	
20"JCX	"	Jersey City, Bailey, 1867 (See Fanning)	72.4	"		Tuberculated		Hering's Kutter	
20"H 25X	"	Hamburg, Iben, 1876	58 to 64	"	25	Heavily incrustated		" "	
20"HOX	"	" Coal Tar "	105 " 110	"	New			" "	
20"DpA	19⅔	Darcy, 1851	108.4 " 117.3	pA	"			" "	
16"D 5pB	16½	Dantzig, Asphalt C't'd. Lampe 1869-71	110.5 " 119.4	pB	5	Good		" "	
16"E 8½X	16	Edinburgh, W.W.	96 & 114	X	8 or 9			Tr. A.S.C.E. V.35 p.278	
15"C 30X	15	Crawley, Jas. Leslie, 1855	86.9	"	30	Incrusted		Hering's Kutter	
12"S 7X	12	Simpson	99.8 & 104	"	Less than 7			Tr. A.S.C.E. V.35 p.278	
12"S 4X	"	"	98	"	" " 4			" "	
12"H12pB	"	Hamburgh, Iben 1876 541' long	75.7 & 76.2	pB	12	Slightly Tuberculated		Hering's Kutter	
12"H14pB	"	" " 1179' "	71 to 80	"	14	Slightly Incrusted		" "	
12"H15pB	"	" " 1808' "	58 " 67	"	15	Heavily Incrusted		" "	
12"H22pB	"	" " 1736 "	45 " 50	"	22	"		" "	
12"PpA	"	Paris, Darcy, 1851	96.9 " 104.8	pA	Old	Carefully cleaned		" "	
16"DAOA	16	Detroit, Coal Tar Army Tangent — Williams Hubbell Fenkell	119.3 " 134.5	A	New	Good	1898	Tr. A.S.C.E. V.47 p.159	
16"DPOA	"	" " Porter "	118.3 " 133.6	"	"	"		" "	
12"DSOA	12	" " So Street "	111.1 " 132.9	"	"	"	1899	" V.47 p.181	
12"DOA	"	" " Straight Pipe "	101.5 " 121.3	"	"	"	"	" " p.187	
36"B.3A	36	Boston " Sherman	136	"	0.3±		1894	" V.44	
36"B1.3A	"	" "	113	"	1.3±		"	" "	
36"B2.3A	"	" "	114	"	2.3±		"	" "	

FIG. 311.—Values of Chezy's C for new and tuberculated cast-iron pipes. Compiled by Engineering Bureau, Board of Water Supply, New York, 1908.

Capacities of riveted steel pipes,* when new, are less than those of cast-iron and welded pipes of same diam., other things being equal. One rule is to make a riveted pipe, for equal capacity, of a diam. greater by twice the height of the rivet heads, but some engineers require a greater difference. In 1910, for



MARK ON DIAGRAM	DIA.	LOCATION	MATERIAL	JOINT	AGE YRS	WT OF EXPT	FIRST USED	SOURCE OF DATA	
72 JT. 1/6 A	72"	Jersey City Water Supply Co	Steel	Taper	1/6	A	Jun. 1903	Official Records	
72 JT. 1 1/3 A	"	"	"	"	1 1/3	"	"	" "	
72 JT. 2 1/3 A	"	"	"	"	2 1/3	"	"	" "	
⊡	"	"	"	"	3	"	"	" "	
48 JT. 0A	48	Cond. 2 E. Jersey W. Co.* abv. Pompton Notch	"	"	New	"	May 1896	Pp 33 53 Herschel's 115 Exps	
48 J Cy. 0B	"	" 1 " "	"	Cylinder	"	B	Apr. 1892	Pp 26.52 " "	
48 Ja Cy. 4A	"	" 1 " " " " "	"	"	4	A	"	" " "	
48 Jb Cy. 4A	"	" 1 " " below " "	"	"	"	"	"	" " "	
42 JT. 0A	42	" 2 " " " " "	"	Taper	New	"	Aug. 1896	Pp. 33,53 " "	
42 KT 0B+	"	Kearney Exten. E. Jersey Water Co.	"	"	"	B+	Jan. 1896	" " "	
36 J Cy 0C	36	E. Jersey Water Co.	"	Cylinder	"	C	Feb. 1892	Pp 32,52 " "	
△	"	" " " "	"	"	4	A	"	Pp. 52 " "	
36 R Cy. 14 C/B	"	Rochester N.Y.	W Iron	"	14	C/B	1876	Pp. 30, 34, 54 " "	
50 BT. 10C	50	Brilliant Pump Sta. to Highland Res. 1 Pitts.	Steel	Taper	10	C	Early part '94	Official Records	
50 Pw 25C	"	" " " " " " "	W Iron		25	"	1879	" "	
108 HCy 5B	108	Holyoke, Mass.	"	Cylinder	5	B	1882±	Pp. 29,34,54 Herschels Exps.	
24 RCy 14 C/B	24	Rochester N.Y.	"	"	14	C/B	1876	Pp. 30,34,54 " "	
38 RCy 0A	38	" "	Steel	"	New	A	Aug 1894	" " " " " "	See Tran. A.S.C.E. Vol.
11 TB.	11			Taper	Good as New	B		Pp. 33,53 " "	40 P. 540 for Corr.
13 TB.	13			"	"	"		" " " " "	
15 TB.	15			"	"	"		" " " " "	
◎	16	Astoria, Oregon		Cylinder	New	"		Pp 34,53 " "	
72 Og Bu 0A	72	Ogden, Utah, Pioneer Elec. Pow. Co.	Steel	Butt	"	A	1897	Tran. Am. Soc. C.E. Vol. 40 P. 471	
33 Po Cy 0	33	Portland, (Ore.) W.W.	"	Cylinder	"	?		" " " " " 26 P 203	On length of 34,176'
35 Po Cy 0	35	" " "	"	"	"	"		" " " " " " "	" " " 39,809'
42 Po Cy 0	42	" " "	"	"	"	"		" " " " " " "	" " " 50,965'
17 TC. 1	17	Texas Creek Cal. C=131.1 for V=20.14	Sheet Iron		1	"		Ham. Smith Hyd. P. 311	" " " 4,439'
72 Og Bu 2A	72	Ogden, Utah, Pioneer Pow. Co.	Steel	Butt	2	A		Tran. Am. Soc. C.E. Vol. 44 P. 311	
48 Na Cy 15A	48	Cond. 1 Newark, above Pompton Notch	"	Cylinder	15	"	Apr 1892	T. Merriman	" " " 23,871'
48 Nb Cy 15A	"	" 1 " below " "	"	"	"	"	"	"	
42 NT. 11A	42	" 2 " " " "	"	Taper	11	"	Aug 1896	"	

A.- Denotes excellent experiments
B.- " good
C.- " poor

†Only 8 yrs since cleaning and painting

*E. Jersey Water Co's conduits became property of the City of Newark about 1897

FIG. 312.—Values of Chezy's C for various types of riveted metal conduits. Compiled by Engineering Bureau, Board of Water Supply, New York, 1908.

Portland, Me., bids were asked on 42-in. cast-iron and 48-in. steel as alternatives for the same line. For lock-bar pipe, capacity 10 to 12 per cent. greater than ordinary riveted pipe is claimed; 5 per cent. is a common figure. At

* See p. 312.

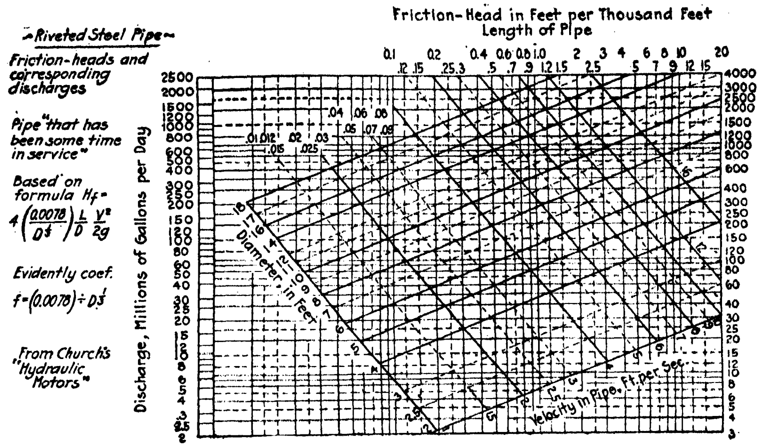


FIG. 313.—Flow in new riveted steel pipes.
(I. P. Church.)[13d]

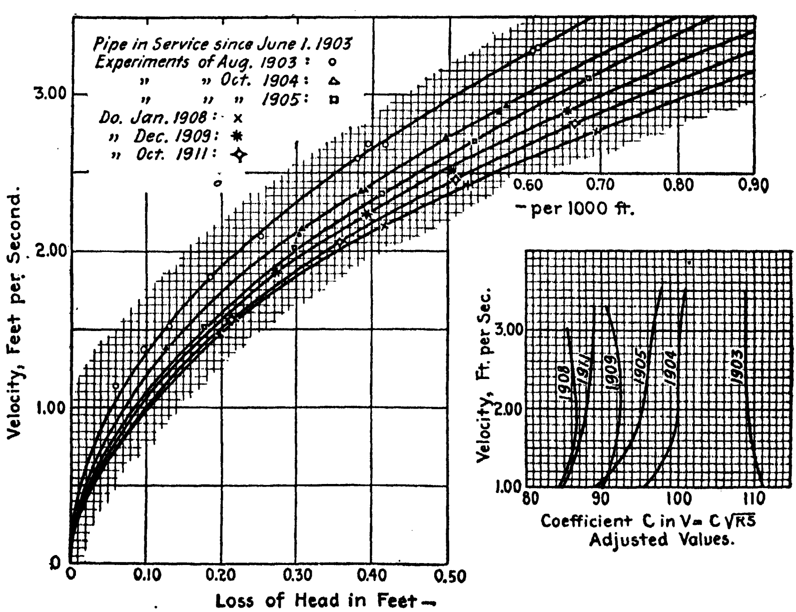
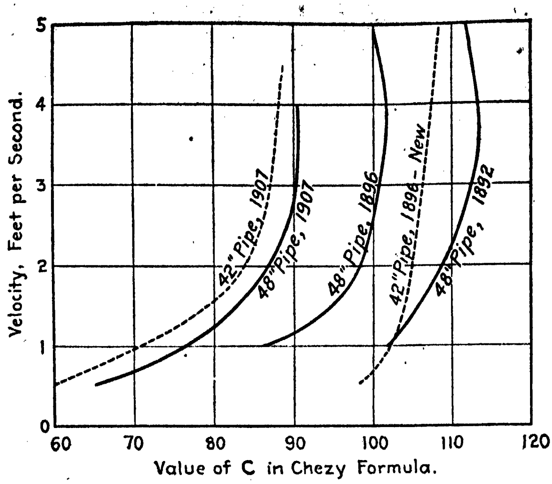


FIG. 314.—Jersey City Water Supply Co. 72-in. riveted steel pipe. Coefficients, velocity and loss of head.*

* About September, 1908, a plant for treating this water with chlorinated lime was installed at Boonton, N. J., and since that date water has been continuously so treated. Immediately following introduction of treatment, large masses of vegetable and other matters were discharged from pipe, and improvement in carrying capacity as indicated by experiments of January, 1908, and December, 1909, is undoubtedly due to this cause.



(Note: Above Values of C are Probable Average Values for Whole Line.)

Fig. 315.—Riveted steel conduits, Newark, N. J. Comparative values of C in 1892, 1896; 1907.

Table 281. Gagings of Northern Section of Conduit II, Rochester, N. Y. Waterworks, between October 1895, and December 1920

Date of experiment	Duration of experiment in hours	Age of conduit in service	38-inch riveted steel pipe				36-in. cast-iron pipe			
			Observed hydraulic grade	Observed mean velocity, ft. per sec.	Computed Williams-Hazen coefficient, C_H	Computed Chezy coefficient $C = \frac{V}{\sqrt{RS}}$	Observed hydraulic grade	Observed mean velocity, ft. per sec.	Computed Williams-Hazen coefficient, C_H	Computed Chezy coefficient $C = \frac{V}{\sqrt{RS}}$
Oct. 17, 1895	5.35	1.27	0.001587	3.88	111	109.35	0.001388	4.20	133	129.5
Oct. 26, 1895	5.22	1.29	0.001614	3.91	111	109.34	0.001507	4.24	128	125.3
Nov. 7, 1895	6.38	1.32	0.001618	3.90	111	109.07	0.001503	4.23	128	125.2
July 28, 1897	10.77	3.04	0.001625	3.81	110	106.1	0.002277	4.13	100	99.2
Nov. 19, 1897	7.08	3.35	0.001533	3.71	108	106.5	0.004854	4.02	65	66.2
June 4, 1898	8.95	3.84	0.001565	3.72	107	105.7	0.004341	4.03	69	70.2
Dec. 21, 1898	8.00	4.44	0.001545	3.71	107	105.9	0.003759	4.03	71	75.3
July 25, 1899	8.00	5.04	0.001535	3.77	109	108.0	0.003446	4.08	79	79.8
Nov. 10, 1899	8.00	5.32	0.001542	3.76	109	107.6	0.003259	4.08	82	81.9
June 26, 1900	6.00	5.96	0.001545	3.74	108	107.0	0.00333	4.06	80	80.7
Dec. 4, 1900	6.00	6.40	0.001560	3.67	106	104.5	0.00324	3.98	80	80.3
June 12, 1901	6.00	6.92	0.001532	3.74	108	107.3	0.00327	4.05	80	81.3
Aug. 5, 1902	7.00	8.06	0.001517	3.69	108	106.4	0.00399	4.00	72	72.5
July 8, 1903	6.00	8.98	0.001438	3.50	105	103.5	0.00279	3.79	82	82.3
July 29, 1904	6.00	10.1	0.00139	3.47	106	104.4	0.00212	3.76	95	93.9
Dec. 8, 1905	6.00	11.3	0.00216	3.75	91	101.9	0.00282	4.07	88	88.0
Dec. 11, 1906	6.00	12.3	0.00184	3.80	100	99.8	0.00302	4.13	85	86.1
Oct. 31, 1907	6.00	13.2	0.00185	3.82	100	99.7	0.00320	4.14	84	91.0
Aug. 18, 1908	6.00	14.0	0.00186	3.91	102	101.8	0.00279	4.24	92	92.0
Oct. 7, 1910	6.00	16.2	0.00184	3.75	99	98.6	0.00261	4.07	91	91.5
Dec. 11, 1911	5.00	17.3	0.00174	3.71	101	100.1	0.00381	4.02	74	74.7
Sept. 30, 1912	6.00	18.1	0.00176	3.77	102	101.2				
Dec. 17, 1913	6.00	19.3	0.00172	3.72	102	101.1				
Oct. 30, 1914	5.0	20.2	0.00150	3.41	100	99.0				
Dec. 22, 1915	5.0	21.1	0.00153	3.37	98	96.8				
Oct. 25, 1916	4.0	22.0	0.00159	3.36	96	94.8				
Oct. 26, 1917	6.0	23.0	0.00161	3.25	92	91.0				
Oct. 29, 1918	5.03	24.0	0.00188	3.59	93	93.1				
Dec. [illegible], 1919	4.00	25.1	0.00151	3.25	95	94.1				
Dec. 2, 1920	4.00	26.1	0.00162	3.27	93	91.6				

Akron, in 1919, bids were taken on 52-in. riveted and 50-in. lock-bar. In New York Board of Water Supply bids, no difference is allowed. The last is sound, as the effect of tuberculation is to nullify any differences in capacities of new pipe chargeable to the friction of the rivet head. Longitudinal joints have little effect on friction losses. Tests by Pacific Gas and Electric Co., on large pipe (8 ft. to 10 ft. 9 in.) led to conclusions that the 10 ft.-9 in. butt-riveted pipe has a slightly lower value of Kutter's n than the 9-ft. pipe with bump joints.[18]

Tests on Lap-riveted Pipe. F. W. Greve,[19] Purdue University, by tests with 4-, 6-, 8- and 10-in. pipes deduced: $H_f = mV^n$; $m = kD^x$; $n = aD^3 + bD^2 + cD + e$. H_f = loss due to friction (in ft. of water); V = true mean velocity (ft. per sec.); D = true diameter (ft).

True diam. inches	m		n	
	With laps	Against laps	With laps	Against laps
4.13	0.115	0.138	1.86	1.91
6.01	0.0843	0.0875	1.91	1.95
8.12	0.0669	0.0739	1.92	1.89
10.21	0.0554	0.0597	1.82	1.80

k, with laps = 0.0488; against laps = 0.0515
x with laps = −0.799; against laps = −0.926

From the data given, m and n may be computed for pipes of any diameter, within reason.

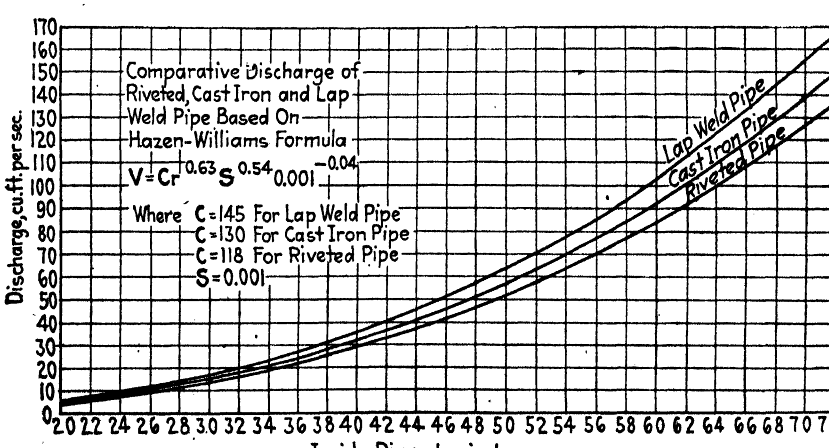


FIG. 316.—Comparative discharges of new metal pipes.*

Penstock Design. Values of n in Chezy's formula recommended as conservative by the Hydraulic Power Committee:[20] lap-welded pipe, bump joints, 0.012; thin-riveted, lap joints, 0.014; pipe, moderate thickness, butt joints, 0.016; heavy pipe, triple-riveted, butt joints, 0.018.

* From Catalog of "National" and Matheson Joint Pipe, National Tube Co.

Corrugated iron pipe, 8 and 10-in., used for culverts, irrigation works, etc., tested by U. S. Dept. Agri.,[21] gave Chezy's $C = 46$ to 55; velocities, 0.6 to 2.8 ft. per sec.

Spiral Riveted Pipe. Experiments by E. W. Schoder at Cornell showed the carrying capacity for 4-in. and 6-in. spiral riveted pipe, to be 6 per cent. in excess of new cast-iron pipe, of the same nominal diam.; perhaps explainable by the fact that spiral pipe is made full diam.

FLOW IN WOOD PIPE*

Kutter's Formula. From experiments by E. A. Moritz[22] on the flow in wood pipes, R. G. Dieck concludes (1) Kutter's n for wood-stave pipe, continuous or sectional, increases with a decrease in velocity (varies inversely with some power of the velocity); for practical purposes its value lies between 0.0100 and 0.0110. (2) There is no great necessity for altering the Kutter formula.

Barnes' Formula. $V = 223.3\ R^{0.660}\ S^{0.586}$. When applied to Ogden and other pipes, this formula gave a variation of V between $+1.5$ and -1.5 per cent.[23]

Norfolk Tests.[69] The new 30-in. and 36-in. conduit comprising concrete, continuous wood-stave, and cast-iron pipe was tested with following results:

Test	Material	Size in.	Length ft.	Flow m.g.d.	C_H
1	Concrete	36	11,464	13.2	140.4
2	Redwood	36	39,580	13.2	138.7
3	Concrete	36	13,071	13.2	137.6
4	Cast Iron	30	4,929	10.2	105.9
5	Redwood	30	4,706	10.2	140.0

Scobey's formulas,[25] based upon all test data known to him, represent within an error of 0.67 per cent. the mean of all experiments, the maximum divergence being 30 per cent.† plus and minus.

$$H_f = 0.419\ \frac{V^{1.8}}{D^{1.17}}$$

$$V = 1.62\ D^{0.65}\ H_f^{0.555}$$

$$Q = 1.272D^{2.65}\ H_f^{0.555}$$

Capacity. Experiments upon old, wood-stave pipe showed some slight increase in capacity after several years' use. No instance is known where a wood pipe kept full of running water has its inner surface rougher after use.

Clean cast-iron pipe would seem to have about 90 per cent. of the capacity of stave pipe, while if seriously tuberculated, a condition which usually prevails after a few years of use, it discharges only about ⅗ as much as a stave pipe of the same size; a steel pipe discharges when clean from 93 per cent., in the case of a 12-in. pipe, to 68 per cent. in the case of a 6-ft. pipe, of the amount which might be expected from stave pipes of the same diam., while if steel pipe is tuberculated

* See also p. 358.
† This large divergence excites the query why the less accurate observations were included.

to an extent that might readily occur in from 10 to 15 years, these discharges may fall to 74 and 54 per cent., respectively.[70] (See also Table 279.)

Ledoux[71] would use Hazen and Williams' Tables for new cast-iron pipe. Repeated tests of a new 24-in. line, 10-mi. long, flowing at 4 ft. per sec. indicated that Chezy's $C = 111$. Scobey's[72] study did not indicate gain or loss in capacity in 15 years; he concludes that wood pipe will convey about 15 per cent. more water than a 10-year old cast-iron pipe or a new riveted pipe, and about 25 per cent. more than a cast-iron pipe 20 years old or a riveted pipe 10 years old. Tests by A. C. Eaton of 96-in. line, 2 years old, at Searsburg, Vt., indicated $C_H = 130$.

FLOW IN MASONRY CONDUITS*

Conditions. Masonry conduits, except pressure tunnels (see p. 291) normally flow less than full and function as open channels (see p. 271). For circular pipes, sewer diagrams similar to Fig. 317, are applicable. Test data on flow in masonry conduits are given by Barnes,† Herschel,‡ and Ganguillet and Kutter.§ Tests on brick intake tunnel at Detroit,[26] 10 ft. diam. and 20 years old, with V varying from 3.13 to 3.51 and averaging 3.34, gave values of C from 116 to 137, and n, 0.0129 to 0.0156, averaging 0.0141. Intake at St. Louis,[27] brick lined except for a granitoid invert, 7.75 ft. high and 9 ft. wide, tested when 27 years old, for average $V = 2.75$ and 3.43, gave Chezy $C = 120.6$ and 119.1. The Northwest Lake and Land Tunnels, Chicago,[28] vary in diam. from 7 ft. 2 in. to 10 ft., and are brick lined. The results of gagings soon after completion in 1902, and in 1917, are given in Table 282.

Table 282. Comparative Results of Tests of Flow in the Northwest Lake and Land Tunnels, Chicago[28]

Average results‖ of four gagings by John Ericson, 1902 and 1903

Section	Dia tu		Ave pe	Ave 10 (1		cto mul
Crib to Kingsbury St.	10	20,483	2.123	0.1288	118.6	0.0145
Kingsbury to Green St.	10	2,216	2.123	0.1108	127.2	0.0135
Green to Keith St.	8	3,328	.470	0.1060	105.5	0.0155
Keith to Springfield Ave.	8	18,855	.373	0.0822	109.1	0.0149
Green to Carroll Ave.	8	2,759	.814	0.1140	119.6	0.0138
Carroll to Central Park Ave.	″ 2″	850 16,647	200 .814	0.1152	119.2	0.0138

Average results of three salt-solution tests, 1919

Section	Dia tu		Ave pe	Ave 10 (1		cto mul
Crib to Chicago Ave. Junc.	10	14,033	3.559	0.333	123.1	0.0142
Chicago Ave. Junc. to Green.	10	8,666	3.145	0.279	122.4	0.0144
Green to Springfield Ave.	8	22,183	2.327	0.203	115.6	0.0144
Green to Central Park Ave.	8	19,856	2.583	0.240	118.5	0.0141

* See also p. 281.
† "Hydraulic Flow Reviewed," (Spon, 1916).
‡ "115 Hydraulic Experiments," (Wiley).
§ Hering & Trautwine translation, (Wiley).
‖ Three gagings only in this case.

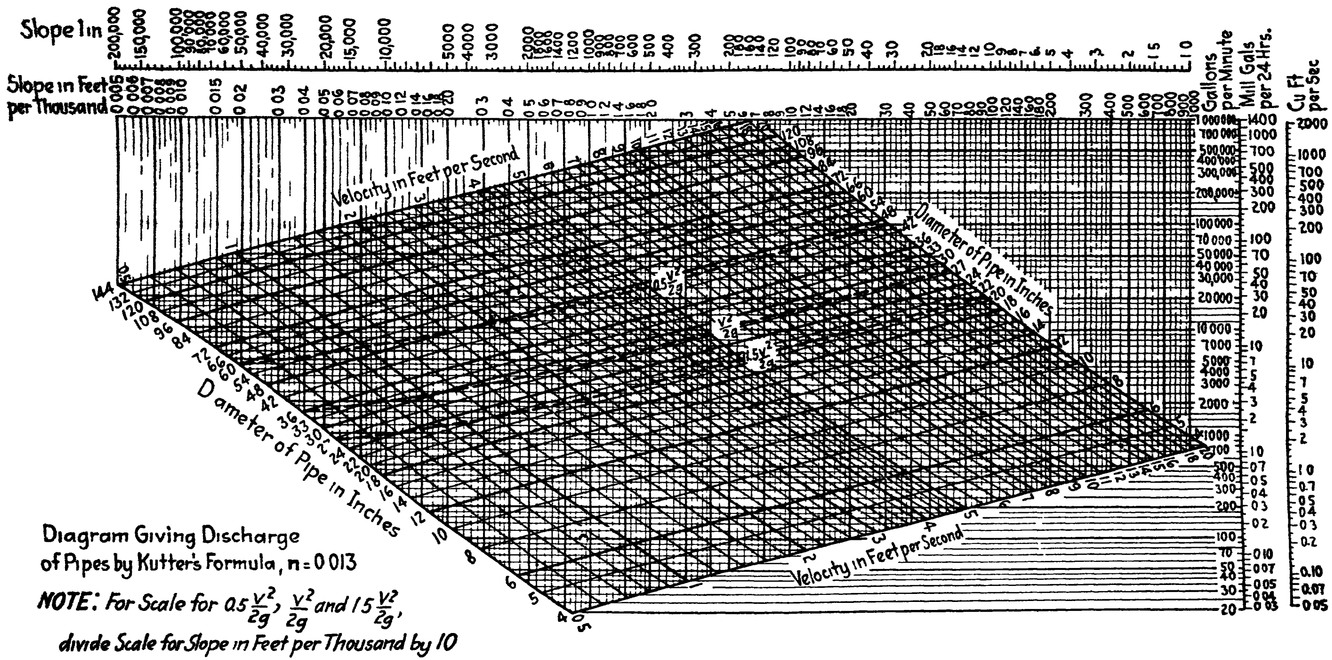


FIG. 317.—Flow in masonry conduits and vitrified pipes.* (*By John H Gregory.*)

(Metcalf & Eddy, "American Sewerage Practice," Vol I, 1914, p 94, McGraw-Hill Book Company, Inc, 1914)

* Some authorities use n = 0 011, others n = 0 012 for concrete lining, see p 375.

Calculations of flow in horseshoe-shaped conduits are facilitated by using tables for ascertaining hydraulic radius.* Hinds has developed tables for ascertaining the critical depth for non-pressure flow; see *E. N. R.*, Oct. 27, 1921, p. 693. The establishing of the critical depth is important in studies for overflow weirs, etc.

Table 283. Flow Coefficients for Concrete-lined Steel Siphons, Catskill Aqueduct[73]

Date of gaging*	Number of siphons gaged	Coefficient in formula $v = C\sqrt{rs}$.		
		Average	Maximum	Minimum
1915	7	164	165	163
1919	13	157†	173	130‡
1920	2	156	169	142

* Made by F. F. Moore, Designing Engineer, New York Board of Water Supply.
† The same 7 siphons that were gaged in 1915 gave in 1919 average 164, maximum 168, minimum 157.
‡ Two siphons near Ashokan reservoir gave 130 and 131, respectively, probably because of the greater fouling by vegetable growths often noted near reservoirs. Excluding these two, average is 162 and minimum is 151.

OTHER LOSSES

Loss of Head in Pipes at Bends.† *Weston Fuller's Formula*[30] (Fig. 318). For 90° bends, $H = CV^{2.25}$, in which H is the excess loss of head over a straight pipe, and C is a coefficient varying with radius of curve; if $R = 0.00$, $C = 0.0135$; if $R = 1.00$, $C = 0.00275$; if $R = 5.0$, $C = 0.00233$; if $R = 20$, $C = 0.00597$. For 45° bends, Fuller suggests $H = 0.75\ CV^{2.25}$, and for $22\frac{1}{2}°$, $H = 0.5\ CV^{2.25}$.

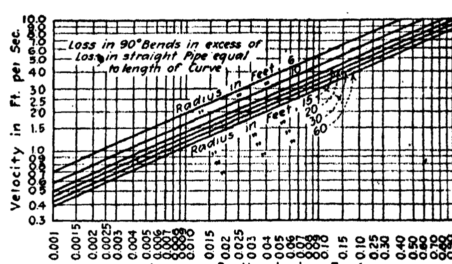


Fig. 318.—Loss in 90° bends (Fuller's formula).‡[31]

University of Texas.[32] Experiments led to the establishment of the formula: $h = 0.00685 \frac{v^{1.77}}{d^{1.275}}$, in which h is the loss of head and v the velocity for straight clean iron pipes ranging in size from $\frac{1}{2}$ to 3 in. when the water has a temperature of about 68°F. and flows at velocities up to 3 ft. per sec. The friction of water at about 68°F. in one standard short-radius steam elbow is $h = 0.0141 \frac{v^{1.96}}{d^{0.26}}$.

* See Metcalf and Eddy, "American Sewerage Practice," Vol. I, pp. 395, 397, 405–407, McGraw-Hill Book Company, Inc.
† See tests of elbows and tees, 1 to 3-in. pipes, *E. N. R.*, May 29, 1924, p. 940.
‡ See other diagrams in E. N., Mar. 2, 1916, p. 413.

Inspection Department of Associated Factory Mutual[33] *Fire Insurance Companies* concluded from tests that losses are:

Long-turn ells* = 0.3 vel. head (or 4 ft. of straight pipe).
Short-turn ells = 1.0 vel. head.
Long-turn tees = 1.0 vel. head.
Short-turn tees = 1.5 vel. head.
6-in. straightway check valve (Pratt & Cady pattern), is equivalent of 50 ft. of 6 in. pipe.
4-in. straightway check valve (Pratt & Cady pattern), is equivalent of 25 ft. of 4 in. pipe.

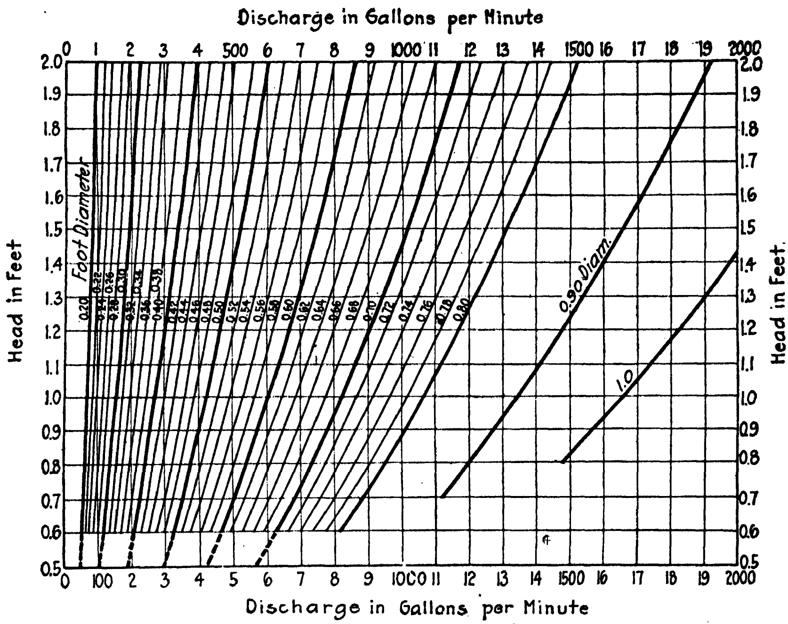


FIG. 319.—Discharges through circular orifices.

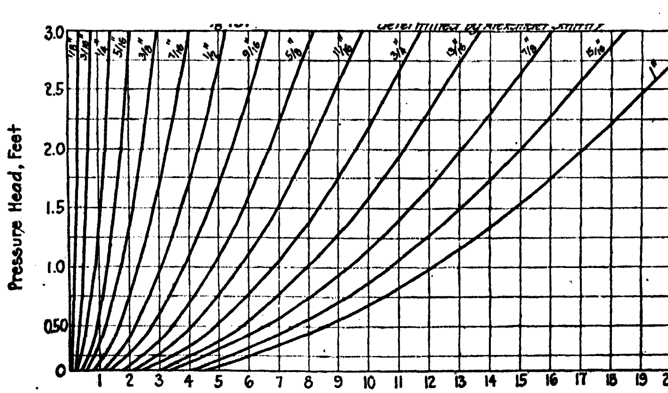


FIG. 320.—Discharges through small circular orifices, $\frac{1}{8}''$ to 1″.
(Hamilton Smith. Jr.)

* Ells are also termed "elbows" and "bends."

Circular Orifice. $\frac{Q}{A} = v = C\sqrt{2gh}$; v = mean velocity, h = head on center of orifice, and A = area. C = coefficient of discharge, averaging 0.60. See Figs. 319 and 320. If the area of channel of approach be small enough so that V_a, velocity of approach, is large,

$$Q = AC\sqrt{2g\left(h + \frac{V_a^2}{2g}\right)}.$$

Rectangular Orifice. $Q = \frac{2LC}{3}\sqrt{2g}\,[h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}]$; L is width, h_1 is head on top, and h_2, head on crest. C varies from 0.60 to 0.67. Bligh[34] gives 0.65 for 2 ft. by 6 in.-opening.

Submerged Tube and Orifice. Rogers and Smith conclude from tests that tube annexed to a square orifice will increase its flow, provided length of tube does not exceed 1.5 times a side of the square;[35] see also *Bull.* 96, Eng. Exp. Sta., University of Ill. 1917.

Discharge through Submerged Orifices.* Tests on irrigation head works[37] indicate that C varies from 0.62 to 0.86 in formula: $Q = CA\sqrt{2g}\;H^{0.454}$.

Flow of Liquids through Cracks.[38] Results of test with filtered water check well with results from formula in Lamb's "Hydrodynamics."† Bronze piston, 6 in. diam., fitted in cylinder 8 in. long with clearance of 0.0018 in. Water temperature, 74°F.

Table 284. Flow of Water through Cracks

Pressure, lbs. per sq. in.	Flow, cu. in. per sec.	
	Calculated	Measured
20	0.21	0.14
40	0.42	0.35
60	0.62	0.61
80	0.83	0.83
100	1.04	1.04
120	1.25	1.28

Flow through Submerged Screens.[39] An investigation on 12-in. square screens, made up of 3-in. by ½-in. pine slats, 1-in. centers, with different conditions of pointing, showed that the flow closely approaches flow through short tubes. $Q = C_oA\sqrt{2g(H_v)}$. A = open area of screens, sq. ft. H_v = difference in water levels above and below screens diminished by the difference in velocity heads. For screen with square corners, $C_o = 0.811$; with downstream face pointed, $C_o = 0.832$; with upstream face pointed, $C_o = 0.985$; with both stream faces pointed, $C_o = 1.032$. Tests on submerged perforated screens, 12 in. square and 0.30 in. thick, gave coefficients corresponding to the flow through orifices in thin plates. With ½-in. holes, spaced 2½ in. on centers, $C_o = 0.640$. With ½-in. holes, spaced ¾ in. on centers, $C_o = 0.758$.[39]

Contractions in an open channel cause a drop in the water surface.[40] Tests of application of Weisbach and d'Aubuisson coefficients led Lane to substitute the general formula:

$$Q = C_QW_2\sqrt{2g}[D_3H_3^{\frac{1}{2}} + \tfrac{2}{3}(H_3^{\frac{3}{2}} - H_2^{\frac{3}{2}})];$$

* See also Professor Unwin's table of coefficients (Bornemann's) at top of page 41, Vol. 14, 11th ed. of *Encyclopedia Britannica*.
† Pp. 553–555.

Q is the quantity in cfs.; C_G, the coefficient; W_2, mean width of stream at most contracted section; D_3, mean depth of channel below the contraction; H_3, the total drop of the water in passing the contraction, or the observed drop plus the head due to the velocity of approach; H_2, the drop of the water surface at the most contracted section including the velocity of approach head.

Gates and Valves.* The loss through a 24-in. gate valve, with conical sections on each end, connecting it to the 42-in. wood-stave pipe line at Atlantic City, was about 0.11 ft., when carrying 5.5 mgd. by test.[41] Tests by Lally[42] of 12-in. valve inserted in 16-in. line with reducers on either side indicated a loss of 0.28 ft. when carrying 5.8 mgd., and 2.12 ft. for 14.7 mgd. Tests at University of Wisconsin on 1.5 to 12-in. valves by Corp and Ruble,[43] indicated that the length of straight pipe of the valve size which will produce the same loss as the valve varies from $\frac{3}{4}$ to 4 ft. for fully open-gate valves, and from 20 to 35 ft. for fully-opened globe valves, the larger figure for globe valves corresponding to the smaller valves.

Butterfly valve tests at University of Pennsylvania indicated that values of K in $h_f = K\frac{V^2}{2g}$ are not constant for a given opening of the valve but vary materially with the velocity in the pipe.[44]

Tests by National Board of Fire Underwriters[33] indicate loss in pressure regulators (and globe valves) = 10 velocity heads. Loss in angle valves = 2 velocity heads.

FLOW OF AIR

Formula for Inflow or Discharge. $Q = C_oAV$

$$V = k(t_a + 273)\sqrt{\frac{n}{n-1}\left(\frac{1}{t+273}\right)\left(\frac{p}{a}\right)^{-\frac{n-1}{n}}\left\{\left(\frac{p}{a}\right)^{\frac{n-1}{n}} - 1\right\}}$$

Q = discharge, cu. ft. per sec.; C_o = coefficient of flow; A = area, sq. ft.; V = velocity, ft. per sec.; p = absolute pressure of internal air; a = absolute pressure of external air; t = temperature of internal air, °C.; t_a = temperature of external air, °C.; k = a constant = 76.344 for dry air.

n = 1.41 for dry air; therefore, $\frac{n}{n-1} = 3.451$; and $\frac{n-1}{n} = 0.291$. For $t = t_o = 17$°C., or about 62½°F., the formula for V (for dry air) becomes:

$$V = 2415\sqrt{\left(\frac{p}{a}\right)^{-0.29}\left\{\left(\frac{p}{a}\right)^{0.29} - 1\right\}}$$

For $p = 14.7$ and $a = 12.7$, $V = 498.2$ ft. For $a = 10.7$, $V = 759$. For $a = 7.35$, $V = 1222$.[45]

To get diam. of inlets of air valves in pipe lines, C_o is assumed at 0.8, as these openings are not in thin plate, but "short pipes" fed by air valves. Hence, since $d^2 = \frac{4Q}{\pi C_o V}$, for $\frac{p}{a} = 2$, $V = 1222$, $d = 0.4336\sqrt{Q}$.†

* See also p. 437.
See "Experiments on the Coefficients of Discharge Under Rectangular Sluice Gates," by Arnold Hartley Gibson. *Proc. Inst. C. E.*, Vol 207, 1918–1919.
† See p. 446 for other methods of design.

PIPE SYSTEM COMPUTATIONS*

Single Cylindrical Clean Iron Pipe.[13f] *No nozzle*, so that V_m, velocity of jet, $= V$, velocity in pipe. Given L, D, and H, find V and Q. For a pipe of this (relative) length, H_F will probably take up a large part of the whole head, H. (Fig. 321.) Assume a trial value of 7 ft. for H_F. This is at the rate of $\left(\frac{7}{0.080}\right) = 87.5$ ft. friction head per thousand. From Diagram 309, $V = 9.1$ ft. per sec. and $\frac{V_m^2}{2g}$ is 1.3 ft.; so that $\frac{\dagger C_o V^2}{2g} = \frac{1}{2} \times 1.3 = 0.65$ ft. To realize the trial value, 7 ft., for H_F, the whole head H would need to be $0.65 + 7 + 1.3 = 8.95$ ft., but this lacks 0.35 ft. of 9.3 ft., the available head. For the next trial, it will probably occasion no great error if the whole of this 0.35 be added to 7 ft. That is, assume $H_F = 7.35$ ft. $\left(\frac{7.35}{0.080}\right) = 92$ ft. friction head per thousand feet. Diagram 309 gives $V = 9.4$ ft. per sec. and $\frac{V^2}{2g} = 1.4$ ft.; $\frac{1}{2} \times 1.4 = 0.7$ ft. Adding, $0.7 + 7.35 + 1.4 = 9.45$ ft., which is so near

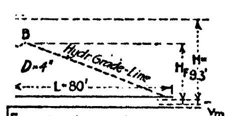


FIG. 321.

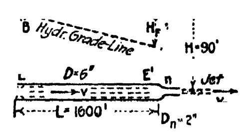


FIG. 322.

9.3 ft, that the second trial may be considered final. Hence, $V = V_m = 9.4$ ft. per sec. and Q (from diagram) $= 0.81$ cu. ft. per sec. N.B. If the jet discharges under water, the results are the same provided the surface of the water in the receiving reservoir is 9.3 ft. below that in the supply reservoir, R. (Both reservoirs large.)

With Nozzle.[47] The jet (Fig. 322) discharges into the air with velocity V_n, to be found. The diam. of the jet is 2 in.; diam. of pipe, 6 in. Solution: V, in pipe, is only $\frac{1}{9}$ of V_n, since the sectional areas are in this (inverse) ratio. From Bernoulli's theorem, p. 751, $H = C_o\dagger\frac{V^2}{2g} + H_F + C_c\frac{V_n^2}{2g} + \frac{V_n^2}{2g}$, where $H = 90$ ft. $C_o = 0.50$; $C_c = 0.05$ (rounded nozzles, or conical play pipe). H is made up of 4 terms, or heads, of which friction head H_F in the pipe is not necessarily a large item, because V is small compared with V_n. Solve by Diagram 309 with successive trial values of some unknown quantity, say V. First assume $V = 5$ ft. per sec., for which friction head would be at the rate of 18 ft. per 1000 ft.; hence, $H_F = \frac{1600}{1000} \times 18 = 28.8$ ft. Since $V = 5$, and $V_n = 9 \times 5 = 45$, $\frac{V^2}{2g} = 0.40$ ft. and $\frac{V_n^2}{2g} = 31.4$ ft. Hence, the two small losses of head would be $\frac{1}{2} \times 0.4 = 0.2$; and $\frac{1}{20} \times 31.4 = 1.57$ ft. $0.2 + 28.8 + 1.57 + 31.4 = 61.97$ ft. (but it should be 90 ft.). **Second trial:** Take $V = 6$ ft. per sec., V_n would be 54 ft. per sec., and the velocity heads

* Church.
† C_o = coefficient of entry loss.

0.56 and 45 ft. respectively; hence, the two small losses of head, 0.28 and 2.25 ft. Now 6 ft. per sec. in a 6-in. pipe implies a friction head at the rate of 25 ft. per 1000 ft. Therefore, $H_F = \frac{1600}{1000} \times 25 = 40$ ft. $0.28 + 40 + 2.25 + 45 = 87.53$ ft. The difference between this and 90 ft. is so small that 6.1 ft. per sec. may be considered final for V; from which follow 55 ft. per sec. for V_n, and 1.2 cu. ft. per sec. for Q. With increasing age the discharge and V would gradually diminish unless the pipe were kept clean. If the entrance E were rounded slightly, a slight increase in Q would result.

Instead of diam. being given, what should be its value that 80 ft. of the total 90 ft. may be available to produce the jet velocity, V_n? 80 ft. $= \frac{V_n^2}{2g}$. $V_n = 71.8$ ft. per sec. At E' the loss of head would be $\frac{1}{20}$ of $80 = 4$ ft., while that at E may be neglected. This leaves 6 ft. for H_F, which is at the rate of 3.75 ft. per 1000 ft. A 2-in. jet at 71.8 ft. per sec. is discharging 1.56 cu. ft. per sec. From Diagram 309, a diam. of 9.9, say 10 in., must be given to the pipe in Fig. 322. This change of design calls for a greater consumption of water (1.56 instead of 1.20) but the "Kinetic Power" (*i.e.*, the kinetic energy of the jet)* $\left(\frac{Q\gamma}{g}\right) \times \left(\frac{V_n^2}{2}\right)$ will be more than doubled: 14.2 horsepower instead of 6.4.

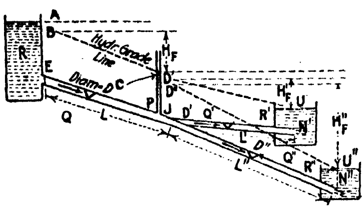


FIG. 323.

Branching Pipes Problem.[13f] The reservoir R supplies R' and R''. The three cylindrical pipes have a junction at J (Fig. 323). When flow is steady, Q (cu. ft. per sec. in main pipe) = sum of Q' and Q'' in the other pipes. H_F, H'_F and H''_F are the respective friction heads. The vertical drop AB is equal to $C_o\dagger\frac{V^2}{2g} + \frac{V^2}{2g}$; and similarly the drop $CD' = C_e\ddagger\frac{V'^2}{2g} + \left(\frac{V'^2}{2g} - \frac{V^2}{2g}\right)$ and $CD'' = C_e'\frac{V''^2}{2g} + \left(\frac{V''^2}{2g} - \frac{V^2}{2g}\right)$. The depths $U'N'$ and $U''N''$ at which the pipes discharge under water are immaterial. There are no nozzles at N' and N''; that is, the jets have the same sectional areas and velocities as the water in the respective pipes. In practice, the velocities in a system of pipes are rarely over 10 ft. per sec., and the pipes are quite long, so that it is sufficiently accurate to neglect the small drops AB, CD', and CD'', and consider that the whole drop from the surface of R to that of R' = sum of the friction heads H_F and H'_F; also the whole drop from R to $R'' = H_F + H''_F$.

Example 1. Determine the proper diam. for the 3 pipes, given $Q' = 13$ and $Q'' = 20$ cu. ft. per sec. (so that Q must equal 33); $L = 30{,}000$ ft., $L' = 10{,}000$ ft., and $L'' = 15{,}000$ ft. The difference of elevation, $R - R' = 60$ ft.; $R - R'' = 85$ ft. Solution for clean cast-iron pipe: Assume one diam., or the friction head H_F of the main pipe; say the latter. Take $H_F = 40$ ft. A steady flow is to take place of 33 cu. ft. per sec., and the friction head

* γ = weight of water, lb. per cu. ft.
† C_o = coefficient of entry loss.
‡ C_e and C'_e = coefficient for loss of head due to elbows, sudden change of section, etc.

is to be at the rate of $\frac{40}{30} = 1.33$ ft. per thousand ft. In Diagram 310, p. 758, the vertical line for 1.33 (interpolating) intersects the horizontal line for $Q = 33$, in a point corresponding to a diam. of 38 in. (among the lines sloping up to the right), while among the lines sloping down to the right, the velocity in a 38-in. pipe would be 4.1 ft. per sec. (not extreme). Deducting the assumed H_F from 60 ft., the corresponding value of $H'_F = 20$ ft., *i.e.*, at the rate of 2 ft. per thousand. From the same diagram, the intersection of the vertical 2 with the horizontal line for $Q = 13$, is a point on the line for a 25-in. pipe; in which the velocity would be 3.8 ft. per sec. (a permissible value). Similarly deducting H_F from 85 ft., $H''_F = 45$ ft., which is at the rate of $\frac{45}{15} = 3$ ft. friction head per thousand. With $Q = 20$, the diagram gives 27.5-in. pipe, with a velocity $= 5.0$ ft. per sec. If H_F had been assumed somewhat greater than 40 ft., a smaller diam. would have resulted for the main pipe, with a higher velocity than before; but larger diam. and smaller velocities in the branch pipes. Results should be sought involving least costs, with sufficient velocities (above 2 ft. per sec.) to prevent deposit of silt.

Example 2. In the above example the diameters of the pipes were sought; but if they were given, and rates of flow to be determined, assume a trial value for Q and find from Diagram 310, the friction head per thousand feet of pipe of the given diam., thence the value of H_F, for the actual length, $EJ = L$. Values of H'_F and H''_F corresponding to H_F are now noted, and corresponding values of Q' and Q'' found from the diagram. $Q' + Q''$ should equal Q. If not, as the result of the first trial, assume a new value for Q; and so on, until the necessary equality is obtained. In the preceding paragraph it is assumed that water flows into R' and R'', and out of R, but if R' is at sufficient elevation, or if the pipe EJ is small, water may flow out of R', as well as out of R. In such case, the summit C would be lower than the surface of R', and $Q + Q' = Q''$. Similar principles and methods apply to any system or network of pipes.

Equivalents of Parallel Pipes. Figure 324 is based on formula $y = \frac{x}{(1 + \sqrt{x})^2}$, where

$$x = \frac{\text{factor of pipes of greater resistance} \times \text{length}}{\text{factor of pipes of less resistance} \times \text{length}}$$

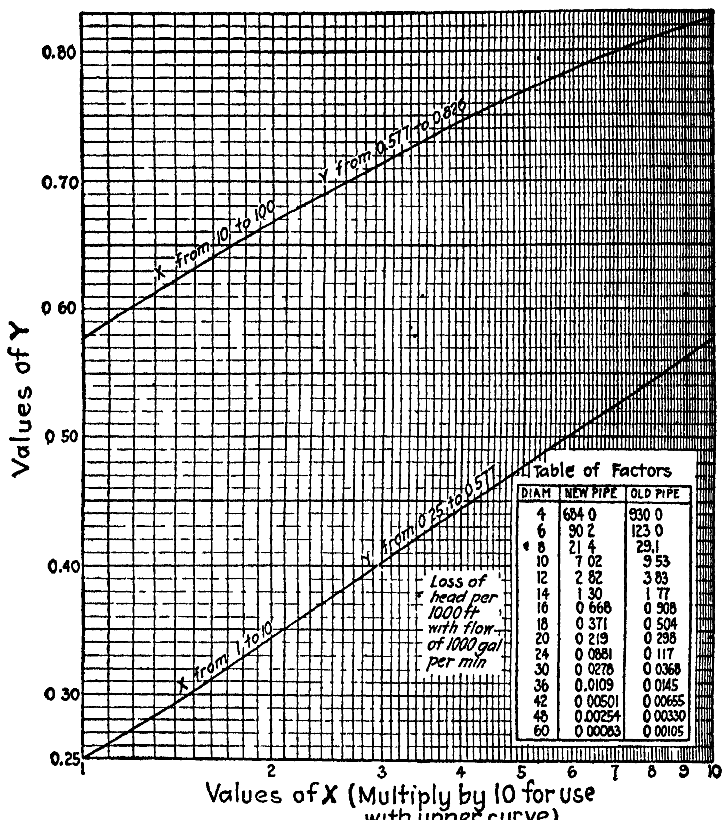
$y =$ coefficient by which f_s must be multiplied to obtain factor of equivalent pipe.

The discharging capacity at any point of a system of connecting pipes may be readily computed by first ascertaining the sizes and lengths of an hydraulically equivalent, single, compound pipe line from the source of supply to the point of discharge. This is accomplished by successively combining a derived equivalent pipe with another pipe of the system. The combinations for the successive computations of the equivalents must be so selected with reference to the physical conditions that total losses of head in the two pipes are the same. For example, consider two pipes leading from a reservoir to the same point in a distribution system; a 10-in. pipe 6,000 ft. long, and an 18-in. pipe 10,000 ft. long, or the problem is the same if these pipes branch below the reservoir; required the diameter of the equivalent single pipe 8,000 ft. long.

For most cases of pipes in use the factors in Fig. 324 for "old pipe" will give results nearer the truth. Then

$$x = \frac{f_l \times \text{length in thousands of ft.}}{f_s \times \text{length in thousands of ft.}} = \frac{9.53 \text{ (from table)} \times 6}{0.504 \text{ (from table)} \times 10} = 11.3$$

y (for $x = 11.3$) $= 0.594$ (from diagram); $0.594 \times 0.504 \times \frac{8000}{1000} = 0.239$ which lies beyond the factor for a 20-in. pipe. Therefore a 24-in. pipe is indicated.



DIAM	NEW PIPE	OLD PIPE
4	684 0	930 0
6	90 2	123 0
8	21 4	29.1
10	7 02	9 53
12	2 82	3 83
14	1 30	1 77
16	0 668	0 908
18	0 371	0 504
20	0 219	0 298
24	0 0881	0 117
30	0 0278	0 0368
36	0.0109	0 0145
42	0 00501	0 00655
48	0.00254	0 00330
60	0 00083	0 00105

FIG. 324.—Equivalent of parallel cast-iron pipes.
(F F Moore)

Division of Flow between Twin Pipe Lines. Suppose that pipes A and B branch at A′ and rejoin at B′ and that they are of different sizes, lengths, and ages. Required the loss of head in the system and the part of the known total flow taken by each pipe. Prepare a diagram with 0 near the center, flows in mgd., plotted as abscissas both to right and left, and friction losses as ordinates. From diagrams similar to Figs. 309 or 310 or from Tables such as Hazen-Williams, on the right and left diagrams, plot curves of losses in each line for various assumed rates of flow. These curves will have a common

point of beginning but will diverge as the assumed flows increase. The intercept on a horizontal line at any point indicates the combined quantity flowing, and the position of the line above the bottom represents the head lost on the ordinate scale. Knowing the total flow, locate the horizontal line intercepting this sum, and read off the head loss, and flow in each line. On Fig. 325, are indicated the calculations for a flow of 3.70 mgd. in the two lines designated.

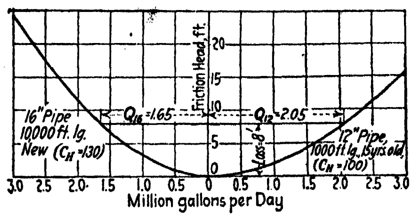


FIG. 325.—Flow in twin cast-iron pipe lines.

Time of Filling a Water Main.* Method of computation is shown by an example and is approximate only. Figure 326 is the profile of a 48-in. pipe line. First air valve controls about 2250 ft. of pipe, and so on, as shown. When water is admitted at standpipe, it first fills nearest depression, and then rises until it overflows next summit. Hence, air displaced at first will probably discharge through all four valves. Until second depression is filled, air will escape at standpipe and through at least the first two air valves. After this, air from section 2 alone will find egress from the first air valve. Now, as

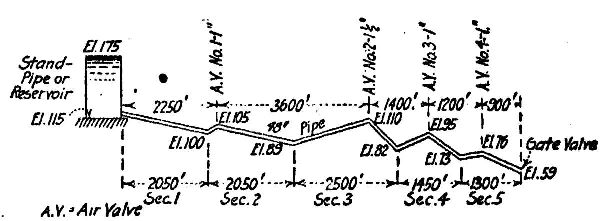


FIG. 326.

water rises in sections 2 and 3, corresponding volumes of air will be expelled at valves 1 and 2. After water overtops the second summit, air displaced from sections 3 and 4 will escape from valves 2 and 3, at least; and possibly some from valve 4. After third depression has been filled and water has risen to third summit, remaining air in sections 4 and 5 must escape by valves 3 and 4.

Assume that pressure on air valves will not exceed 10 lb. gage. Then

$$\frac{\text{gage pressure}}{\text{atmospheric pressure}} = \frac{24.7}{14.7} = 1.68 \text{ and } V = 1049\dagger$$

For 1-in. air valves, $A = 0.7854$ sq. in. $= 0.00545$ sq. ft.
For 1½-in. air valves, $A = 1.767$ sq. in. $= 0.01227$ sq. ft.

* For precautions, see p. 412.
† Air-flow formula, p. 772.

Since passage through air valve is not an "orifice in thin plate," but more nearly a short pipe, take $C_o = 0.8$. (C_o = discharge coefficient)

Q for 1-in. air valve $= 0.8 \times 0.00545 \times 1049 = 4.58$ cu. ft. per sec.
Q for $1\frac{1}{2}$-in. air valve $= 0.8 \times 0.01227 \times 1049 = 10.30$ cu. ft. per sec.
100 ft. of 48-in. pipe has a content of 1257 cu. ft.

Since first depression is near first summit, the grade between is steep, and the cross-section of the pipe at this lowest point will soon be filled. As indicated in Fig. 327, suppose content of this depression equal to that of 125 ft. of pipe, or $1.25 \times 1257 = 1571$ cu. ft. To fill it will require $1571 \div 18 = 87$ sec. $= 1.5$ min. if water be admitted at rate of 18 cu. ft. per sec., found necessary by computations below to discharge air at above rates. If pressure of 10 lb. be maintained at air valves 1 and 2, air will flow from them at $4.58 + 10.30 = 14.88$ cu. ft. per sec. Sections 2 and 3 as far as second summit contain together $36 \times 1257 = 45{,}252$ cu. ft. Neglecting slight difference in elevation between first and second summits, it will take 3037 sec. = 50.6 min. to fill the main to the second summit. For approximate computation third depression may be considered equivalent to first. This air will escape from valves 2 and 3 in $1571 \div 14.9 = 105.5$ sec. $= 1.8$ min. Remaining air in section 4 below third summit, say in $650 - 50 = 600$ ft., is 7542 cu. ft., and will pass from air valve 3 in 1640 sec. = 27.3 min. Assuming fourth depression, also equivalent to first, it will take $1571 \div 9.2 = 171$ sec. $= 2.9$ min. to get this air out through valves 3 and 4. This leaves about $1200 + 900 - 125 = 1975$ ft. of main, containing 24,826 cu. ft. of air to be discharged through two 1-in. air valves, which will require 2698 sec. = 45 min.

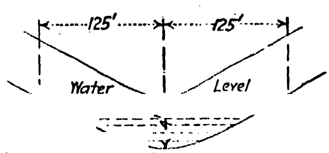


Fig. 327.

There is left part of section 3 between second summit and third depression, which is above the elevation of the third summit, about 400 ft., containing 5028 cu. ft. and requiring 488 sec. = 8.1 min. to fill. Section 1 has been filling meanwhile, but, probably, 900 ft. remain empty. Water may be admitted here more rapidly, say at 30 cu. ft. per sec., provided air has quite free egress at standpipe, as would be the case if standpipe and main were being filled together, or if there were an ample air valve at this end of the main; $9 \times 1257 \div 30 = 380$ sec. $= 6.3$ min. will complete the filling of the section. Total time for filling main from standpipe to gate valve will be, therefore, the sum of these partial times, 143.5 min., or about 2 hr. and 24 min.

To maintain conditions named above while filling the main, water should be admitted at the standpipe as follows: first 54 min. at 18 cu. ft. per sec.; next 27 min. at 10 cu. ft. per sec.; next 48 min. at $9\frac{1}{4}$ cu. ft. per sec.; next 8 min. at $10\frac{1}{2}$ cu. ft. per sec. (or say 10 cu. ft. per sec. for 80 min.); last $6\frac{1}{2}$ min. at 30 cu. ft. per sec. If rates of admission of water be materially increased for any period, pressure on air valves would probably exceed 10 lb. per sq. in., and if valves were so constructed as to close when internal pressure exceeded this amount, there would be immediate danger of serious water ram. Hence, measure the water or place a pressure gage near or beyond first valve.

Pipe Siphons. (True Siphons.)* For drawing water from reservoir, over bank, etc. At E. London, Cape Colony, a 12-in. pipe gave following results on test for liberation of air (see Fig. 328); capacity of air vessel, 650 gal.; difference of elevation between water in reservoir and air vessel, maximum 15 ft., minimum 6 ft., mean, 10.5 ft.; water discharged Feb. 12 to Mar. 22, 1906, 14 mg.; air liberated reduced to atmospheric pressure, 535 gal. = 0.0038 per cent., under mean vacuum of 10.5 ft.; mean temperature 70°F. (atmosphere). Water was drawn from an impounding reservoir and

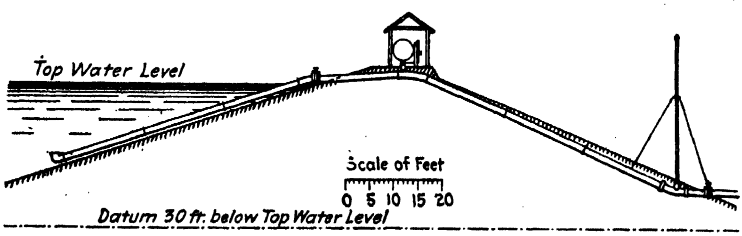


FIG. 328.—Siphon outlet from reservoir.[48c]

contained about 4 per cent. dissolved gases. Water was discharged at rate of 0.36 mgd. = 0.56 cfs.[48e]

A 68-in. riveted steel pipe had, see Fig. 329, an air receiver at delivery end, from which a small steam jet extracted air; but a large quantity of air liberated was entrained and carried out with the discharged water. With 6-in. difference of water levels, delivery was very approximately 43 mgd. = 67 cfs.[48d]

Air in water: rate of rise of air bubbles through water. In flowing through gate houses and other structures, under certain circumstances, water

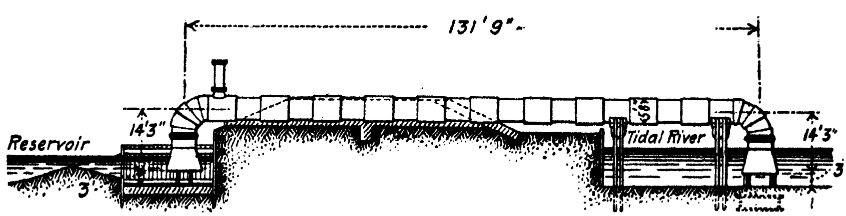


FIG. 329.[48d]

entrains air, and it is sometimes necessary to remove excess air before permitting water to flow on into an aqueduct or pipe. In connection with design of an air remover for Catskill aqueduct[49] to determine rate at which air would probably separate from water, simple experiments were made (1909), see Fig. 330, on rate at which air bubbles would rise through water. A wooden tank 6 in. square inside, 11 ft. 4 in. high, with panes of glass at intervals on one side, and a small pipe near bottom for admitting air under pressures up to 5 lb. was used. The air pipe was horizontal with end capped, one hole

* See Brann & Sherman, "Operation of a True Siphon on a Main Supply Pipe," *J. N. E. W. W. A.*, Vol. 35, 1921, p. 36.

$\frac{1}{16}$ in. diam. in top of pipe at vertical center line of tank; air was controlled by a cock. Water temperature was 70°F., air 72°F.

Water discharged from a hose immersed about 5 ft. in water liberated air in bubbles of various sizes, including a quantity of very small ones $\frac{1}{64}$ in. and less. No attempt was made to measure rate of rise, but it was noted that the smaller bubbles rose very slowly. Water was discharged from $\frac{1}{4}$-in. nozzle on to the water surface in the tank. It carried air with it to a depth of 6 to 8 in. in form of bubbles from $\frac{1}{8}$-in. diam. to very small ones. There seemed to be a distinct division of bubbles into two groups, those about $\frac{1}{16}$ in. in diam. and over, and those about $\frac{1}{64}$ in. and under. There were very few bubbles of diam. between $\frac{1}{16}$ and $\frac{1}{64}$ in. Pressure at nozzle probably about 15 lb.

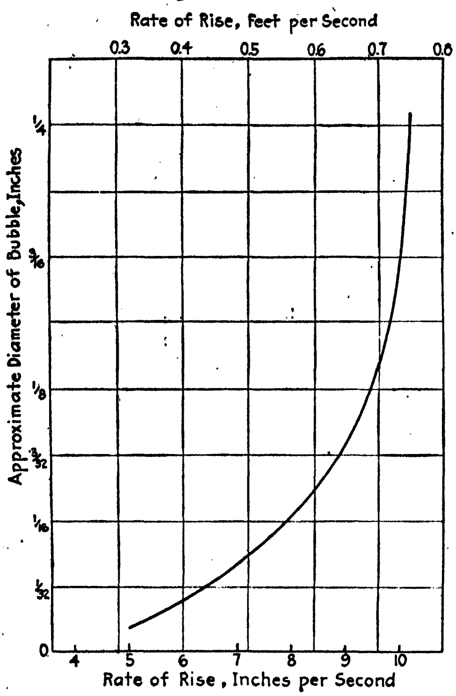


FIG. 330.—Velocity of air bubbles rising through water. Experiments at Bd. of Water Supply Laboratory, New York, Aug., 1909.

The velocity of "commotion" bubbles (about $\frac{1}{4}$ in. diam.) is about 9 ft. per sec. The velocity varies as the square root of the diam., and the diam. increases as the bubble approaches the surface due to the lessening pressure. "Elimination" bubbles are seldom over 0.01 in. diam. at any stage of their ascent, and cannot rise at a rate greater than 2 in. per sec.[50]

Experiments made with glass tubes 5 in. in diam. showed a velocity of 8 to 10 in. per sec. for $\frac{3}{8}$-in. bubbles. The velocity does not depend on the depth of water.

Table 285. Rate of Rise of Air Bubbles through Water

No. of expts. in av.	Estimated size of bubble, in.	Av. time, minutes, consumed in rising		Remarks
		5 ft.	10 ft.	
6	$\frac{1}{64}$	0.24	0.47	Bubbles liberating themselves naturally from orifice in top of air pipe.
2	$\frac{1}{32}$	0.17	0.30	
4	$\frac{3}{64}$	0.15	0.31	
3	$\frac{1}{16}$	0.14	0.27	
4	$\frac{1}{8}$	0.12	0.24	
3	3	0.05	0.11	Large hemispherical bubbles, made by releasing large quantity of air quickly, rising at head of large groups of smaller bubbles.
6	4	0.04	0.10	
1	5	0.05	0.10	
7	$\frac{1}{64}$	0.24	0.47	Time is that required, after liberating large quantity of air, to clear water of all bubbles larger than size stated.
4	$\frac{1}{32}$	0.20	0.41	
2	$\frac{1}{16}$	0.17	0.34	
5	$\frac{1}{8}$	0.13	0.24	
2	$\frac{1}{4}$	0.11	0.22	

WATER-HAMMER IN PIPES*

Investigations.† This subject is too complex for brief statement. Engineers having important problems should consult the authorities, some of whom are mentioned below. Useful formulas are given by Prof. W. F. Durand, in the "Hydraulics of Pipe Lines," Constable & Co., London, 1921, and in *E. N. R.*, Dec. 23, 1920, p. 1212, "Water Hammer in Pipe Lines," but the formulas cannot be separated from their context. They are based on the Joukovsky,[54] or acoustic wave theory.‡ It has been customary to assume for simplicity various restrictive conditions, but omitting thereby factors which have important bearing on numerical results. As King[4b] points out: "Discrepancies of from 100 to 220 per cent. are possible." Durand gives an analytical treatment of the problem including conditions as they exist in actual practice—imperfect reflection at the valve, friction in the pipe line, velocity head, any time rate of valve area change, and loss of energy through the discharge valve. Further experimental knowledge is needed.

Importance. Allowance for water-hammer is essential in designing pipe systems; extra thickness is required for cast-iron pipes (see p. 382). Many breaks in lines and pumps are on record; *e.g.*, the wheel of a 4-in. centrifugal pump was cracked by water-hammer in Arizona.[51]

Stresses in Pipe.[13c] When water flows through a very long pipe, sudden closing of valves, especially the last quarter of travel, tends to cause high momentary bursting pressures, or water-hammer; extreme pressure would be occasioned by instantaneous closing. If the pipe does not move lengthwise, kinetic energy of water will exhaust itself in compressing water and distending walls of pipe. Water has only one modulus of elasticity, "Bulk Modulus," or E_b. For pressures below 1000 lb. per sq. in. (and at ordinary temperatures) E_b may be taken as 294,000 lb. per sq. in., or 42,400,000 lb. per sq. ft. Neglecting distention of pipe,§ fluid pressure induced by compression of the water,

$$P_r = V\sqrt{\frac{E_b w}{g}} \text{ or } P_r(\text{in lb. per sq. in.}) = 63 \times V(\text{in ft. per sec.})$$

w = unit weight of water, and V = initial velocity. "Excess pressure" is proportional to original velocity V of water in pipe; t = thickness of pipe wall, and r = internal radius of pipe. If E is modulus of elasticity of metal of pipe, distention of latter being considered, "excess pressure" tending to burst pipe is

$$P_r = V\left(\sqrt{\frac{wEE_bt}{g(tE + 2rE_b)}}\right).$$

Surges in Pipe Lines. The increased stress in lb. per sq. in. of metal, S_1, due to the sudden stoppage of water $= 307V\sqrt{\frac{d}{t}}$, where V = velocity of flow in ft. per sec. in the pressure pipe just before the interruption; t = maximum thickness of pipe, in.; d = nominal diam. of pipe, in. Example:

* Sometimes called water ram or shock.

† See also "Pulsations in Pipe Lines, as Shown by Some Recent Tests," by H C Vensano, *T. A. S. C E*, Vol 82, 1918, p 185. "Pressures in Penstocks Caused by the Gradual Closing of Turbine Gates," by Norman R Gibson, with discussion, *T A S C E*, Vol 83, p 207, 1919-1920.

‡ See *Memoirs*, Imperial Academy of Science, St Petersburg, 1897, Vol 9; also Allievi, in Annali della Società degli Ingegneri, Rome, Vol. 17, 1902

§ For very high pressure see *E. N*, Oct 4, 1900, p 236

In a 60-in. pipe, 0.5 in. thick at the lower end, what is the fiber stress due to the sudden stopping of water flowing at a velocity of 5 ft. per sec.? $\frac{d}{t} = 120$, and $S_1 = 17{,}000$ lb. per sq. in. The sum of the initial stress, S, due to the static head, and S_1, gives the required stress.

Excess pressure, in ft. head $= 1415V\sqrt{\frac{t}{d}}$. This excess pressure is independent of the length of pipe, because each foot of pipe which adds to the energy of the water also adds to the length of pipe on which work is done. These equations are for instant closure of gates, and show the necessity for slow closing. If the energy imparted to the pipe is neglected, an assumption on the side of safety, the rise in pressure due to the gradual closing of the valves $= 0.0135La$. In this case the rise in pressure is directly proportional to the length of pipe, L, and to the rate of bringing the water to rest. In other words, a the rate of acceleration, $= \frac{\text{velocity}}{\text{time}}$[52]

Practical Results. Water-hammer is the more severe the quicker the closing, the higher the velocity of the water, and the greater the length of the moving column of water. Sudden closing of an automatic balanced valve in a short 42-in. steel pipe, at Springfield, Mass. filters, static head 25 ft.,

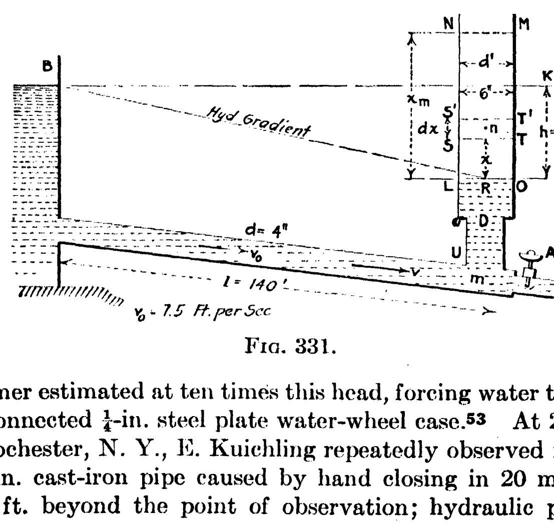


FIG. 331.

caused hammer estimated at ten times this head, forcing water through riveted joints in a connected ¼-in. steel plate water-wheel case.[53] At 2.53 miles from reservoir, Rochester, N. Y., E. Kuichling repeatedly observed increased pressures in 24-in. cast-iron pipe caused by hand closing in 20 min. of a 24-in. valve, 2100 ft. beyond the point of observation; hydraulic pressure, valve open, 40.2 lb.; velocity 3.45 ft. per sec.; static pressure, valve shut, 49 lb. Last 4 min. of closing, pressure rose to maximum of 65 lb., then oscillated rapidly a few seconds 40 to 65 lb., then gradually reduced in 2 min. to 42 to 57 lb., and in 8 min. more to 46 to 52, with increasing intervals between pulsations; at end of 15 min. pressure varied little from 49 lb. Hydraulic elevators, locomotive standpipes, and similar devices are prolific causes of water-hammer. Air chambers, relief valves, and open standpipes extending above normal hydraulic gradient are means for preventing damage. Slow closing of valves is best prevention. Steel cylinders 5 ft. diam. and 15 ft.

long mounted in a horizontal position just outside the pump house on the 37-mile pumping line for El Paso & South Western Ry. prevented water-hammer; the pressure variation was gaged as less than 2 lb.[55] under operating conditions.

Surge in Hydraulic Standpipe.[56] In each experiment,* steady flow was first secured with LO 11 ft. below BK,† corresponding hydraulic grade line being BR. Let $\frac{1}{4}\pi d^2$, sectional area of main pipe, be denoted by F, and that of standpipe (at LO and above), viz., $\frac{1}{4}\pi d'^2$, by F'. Let the valve at A be instantaneously closed. Surface of water in the standpipe immediately begins to rise above its initial position LO, and velocity, v, of the water in the main pipe begins to diminish from initial value, $v_o = 7.5$ ft. per sec. Unsteady flow now sets in; what value of v will correspond to any value of x (height of surface ST in standpipe, above LO), during upward "surge;" *i.e.*, find v as a function of x and finally determine greatest height $x_m = \overline{OM}$ of surge, the water in both pipes having then come to rise, for an instant. Let γ denote weight of a cu. ft. of water, and g acceleration of gravity. Apply Principle of Work and (Kinetic) Energy, as governing the motion of an assemblage of rigid bodies, to the movement, in time increment, dt, of all particles of water in supply pond, main pipe, and standpipe; their aggregate weight being G lb. Equating work of working force G to friction work plus change of kinetic energy, all forces besides gravity and friction being either neutral or self-cancelling (atmospheric pressure)

$$\gamma F'dx(h - x) = \gamma \frac{Fl}{g} vdv + \gamma \frac{F'x}{2g}\left[1 + \frac{4fl}{d}\right]\ddagger \qquad (1)$$

Dividing through by $F'\gamma$, also denoting $\frac{1}{2g}\left[1 + \frac{4fl}{d}\right]$ by A, and $\frac{F}{F'}\cdot\frac{l}{g}$ by B,

$$[h - x - Av^2]dx = Bvdv \qquad (2)$$

$$\text{Integrating, } Av^2 = h - x + \frac{B}{2A}\left[1 - \epsilon^{-\frac{2A}{B}x}\right] \qquad (3)$$

which is the desired relation giving velocity v in the pipe for any value of x during the (first) upward motion, or "surge," of the surface ST of the water in the standpipe. Position reached by this surface at end of first surge is found by putting v = zero in (3) and solving for x, which is then called $x_m = \overline{OM}$ in Fig. 331.

$$x_m = h + \frac{B}{2A}\left[1 - \epsilon^{-\frac{2A}{B}}\right. \qquad (4)$$

This transcendental relation can be solved for x_m only by trial; not tedious, since the value of the bracket is not greatly affected by changes in x_m as successive assumptions are made. ϵ is 2.71828, Naperian base.

Applying Eq. (4) to experiment with 6-in. standpipe ($d' = 6$ in.), we have (since $h = 11$ ft., $= Av_0^2$ and $v_0 = 7.5$ ft. per sec.), $A = 0.1955$; while

$$B = \frac{F}{F'}\cdot\frac{l}{g} = \frac{(4)^2}{(6)^2}\cdot\frac{140}{32.2} = 1.932;$$

both values involving ft. and sec. as units.

* Experiments, 1909–1910, Hydraulic Lab., Cornell University, by H. H. Conway and F. H. Storey.
† Fig. 331.
‡ f = friction factor, see p. 754.

Hence

$$\frac{B}{2A} = 4.94 \text{ ft.; while } \frac{2A}{B} = 0.2024.$$

With 20 ft. as first trial value for x_m,

$$\epsilon^{-\frac{2A}{B}x_m} \text{ becomes } \epsilon^{-0.2024 \times 20} = \epsilon^{-4.048}$$

and hence it is reciprocal of number whose natural logarithm ($\log_\epsilon$) is 4.048, which is 57.

From Eq. (4), $11 + 4.94\left[1 - \frac{1}{57}\right] = 15.85$ ft.,

which is much smaller than assumed 20 ft.

Assuming 16 ft. for x_m, right-hand member of (4) = 15.7; assuming 15.7 ft. for x_m, from (4)

$$x_m = 11 + 4.94\left[1 - \frac{1}{2}\right] = 15.74 \text{ ft.}$$

Result of actual experiment was $x_m = 16$ ft.

Experiment with 4-in. standpipe, $h = 11$ ft. and $v_0 = 7.5$ ft. per sec.; A, as before $= 0.1955$; but since now diam. of standpipe is equal to that of main pipe, so that $F = F'$,

$$B = \frac{140}{32.2} = 4.348.$$

$$\frac{B}{2A} = 11.12 \text{ ft.; and } \frac{2A}{B} = 0.0900.$$

Assume $x_m = 25$ ft., right-hand member of Eq. (4) becomes

$$11 + 11.12\left[1 - \frac{1}{9.4}\right] = 20.93 \text{ ft.}$$

Finally value 20.3 ft. proves sufficiently close. Value found by experiment was 19.8 ft.

For practical use, transform Eq. (4) by writing $\frac{1}{K} = \frac{2A}{B}$ in exponent inside bracket, while restoring original meaning of A and B *outside* bracket,

$$x_m = h + \frac{F}{F'} \cdot \frac{l}{1 + \frac{4fl}{d}}\left[1 - e^{-\frac{x_m}{K}}\right] \tag{5}$$

(Note that the coefficient outside the bracket = K.)

In solving Eq. (5) for x_m in any actual case of a water-power plant with a long supply pipe, a fair value for the first approximation may usually be obtained by neglecting second term in bracket, since value is generally small compared with unity. As to *time* occupied in first upward surge, observed time in case of 6-in. standpipe was 10 sec.; and for 4-in. standpipe, 5 sec. A relation holding between v and x for any instant during return motion (first *downward* surge) of water in standpipe (with corresponding "backward" motion of water in main pipe) can easily be established by use of foregoing methods, it being remembered that initial conditions of this downward surge

are final conditions of first upward surge. Another upward surge now follows, and so on; but each succeeding surge is shorter in range than that immediately preceding until finally water in standpipe comes to rest with summit on level with that of lake surface.[*]

WEIRS[†]

Bazin's Weir Formula.[‡] Bazin's results per lin. ft. of weir, for weirs with sharp crests,[§] without end contractions, are approximated by:

$$Q = c\left[1 + 0.55\left(\frac{H}{D+H}\right)^2\right]H^{\frac{3}{2}}\sqrt{2g}$$

$$c = 0.405 + \frac{0.00984}{H}$$

or more roughly,

$$Q = c'H^{\frac{3}{2}}\sqrt{2g}$$

$$c' = 0.425 + 0.21\left(\frac{H}{D+H}\right)^2$$

D and H are in ft., Q in cfs. (Fig. 332).

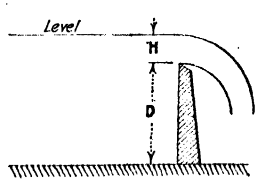


FIG. 332.

Francis' weir formula (for sharp-crested weir with two end contractions, and no correction for velocity of approach); $Q = 3.33\,(L - 0.2H)H^{\frac{3}{2}}$. Q = cfs., H = head and L = length, in ft. Studies since 1852 have shown that this formula may be in error 5 or 10 per cent. It is extensively used on account of its supposed simplicity. (See discussion in King's "Handbook of Hydraulics," McGraw-Hill Book Company, Inc., 1918, p. 67.) Schoder[57] warns that the formula should not be used unless $L > 2h$. See Steward and Langwell, *T. A. S. C. E.*, Vol. 76, 1913, p. 1045.

* See also *T. A. S. C. E.*, Vol. 78, 1915, p. 760, R. D. Johnson.
† See Herschel's studies on Hollowcrest weirs, *J. A. S. M. E.*, Feb., 1920, p. 83.
‡ For verification see paper by Nagler, *T. A. S. C. E.*, Vol. 83, 1919–1920, p. 105.
§ See page 791 for flat crests.

Table 286. Rectangular Weir Discharges (Two Full End Contractions)

Gallons per Minute for Heads Measured to 1/8". Between 100 and 1000, Figures to Nearest Unit. Above 1000, Figures to Nearest 10.

Head	0"	1/8"	1/4"	3/8"	1/2"	5/8"	3/4"	7/8"
	Weir Length, 12 Inches							
0"	....	1.57	4.45	8.25	12.6	17.6	23.1	29.0
1"	35.3	42.1	49.2	56.7	64.4	72.4	80.8	89.4
2"	98.3	108	117	126	136	146	156	167
3"	177	188	199	210	222	233	245	256
4"	268	280	293	305	317	330	343	356
5"	369	382	395	408	421	435	448	462
6"	475	489	503	517	532	547	560	574
	Weir Length, 24 Inches							
0"	....	3.14	8.92	16.6	25.4	35.3	46.5	58.4
1"	71.3	85.0	99.5	115	130	147	164	182
2"	200	219	238	258	278	299	320	342
3"	364	387	410	433	457	482	506	531
4"	556	582	608	634	661	688	715	743
5"	771	799	827	856	885	914	944	974
6"	1000	1030	1060	1100	1130	1160	1190	1220
7"	1250	1290	1320	1350	1380	1420	1450	1480
8"	1520	1550	1590	1620	1660	1690	1730	1760
9"	1800	1830	1870	1900	1940	1980	2010	2050
10"	2080	2120	2160	2200	2230	2270	2310	2350
11"	2380	2420	2460	2500	2540	2570	2610	2650
12"	2690	....	....	....	....	....	....	....
	Weir Length, 36 Inches							
0"	....	4.71	13.4	24.9	38.1	53.0	69.8	87.9
1"	107	128	150	173	197	221	247	274
2"	302	330	359	390	420	452	484	517
3"	551	586	621	656	693	730	767	805
4"	844	883	923	963	1000	1040	1090	1130
5"	1170	1220	1260	1300	1350	1390	1440	1490
6"	1530	1580	1630	1670	1720	1770	1820	1870
7"	1920	1970	2020	2070	2120	2170	2230	2280
8"	2330	2390	2440	2490	2550	2600	2660	2710
9"	2770	2820	2880	2940	2990	3050	3110	3160
10"	3220	3280	3340	3400	3460	3520	3570	3630
11"	3700	3760	3820	3880	3940	4000	4060	4120
12"	4180	....	....	....	....	....	....	....
	Weir Length, 48 Inches							
0"	....	6.28	17.9	33.2	50.9	70.8	93.4	117
1"	143	171	200	231	263	296	331	366
2"	403	442	481	521	563	605	648	693
3"	738	784	831	879	928	978	1030	1080
4"	1130	1180	1240	1290	1350	1400	1460	1520
5"	1570	1630	1690	1750	1810	1870	1940	2000
6"	2060	2120	2190	2250	2320	2390	2450	2520
7"	2590	2650	2720	2790	2860	2930	3000	3070
8"	3150	3220	3290	3360	3440	3510	3590	3660
9"	3740	3810	3890	3970	4040	4120	4200	4280
10"	4360	4440	4520	4600	4680	4760	4840	4920
11"	5010	5090	5170	5260	5340	5420	5510	5590
12"	5680	....	....	....	....	....	....	....

Table 287. Rectangular Weir Discharges without End Contractions,* Francis Formula

Depth of water on weir (in.)	Discharge per lin. ft. of weir		Depth of water on weir (in.)	Discharge per lin. ft. of weir		Depth of water on weir (in.)	Discharge per lin. ft. of weir		Depth of water on weir (in.)	Discharge per lin. ft. of weir	
	Cu. ft. per sec.	Gals. per min.		Cu. ft. per sec.	Gals. per min.		Cu. ft. per sec.	Gals. per min.		Cu. ft. per sec.	Gals. per min.
1	0.0801	35.95	8	1.8126	813.5	15	4.6538	2089	22½	8.550	3837
1¼	0.1120	50.27	8¼	1.8983	852.0	15¼	4.7673	2140	23	8.836	3966
1½	0.1472	66.06	8½	1.9852	891.0	15½	4.8880	2194	23½	9.126	4096
1¾	0.1854	83.21	8¾	2.0736	930.6	15¾	5.0071	2247	24	9.419	4227
2	0.2266	101.70	9	2.1629	970.7	16	5.1268	2301	24½	9.715	4360
2¼	0.2703	121.31	9¼	2.2537	1011.5	16¼	5.2477	2355	25	10.013	4494
2½	0.3166	142.09	9½	2.3456	1052.7	16½	5.3691	2410	25½	10.315	4629
2¾	0.3654	164.00	9¾	2.4389	1094.6	16¾	5.4916	2465	26	10.620	4766
3	0.4162	186.79	10	2.5332	1136.9	17	5.6149	2520	26½	10.928	4904
3¼	0.4693	210.64	10¼	2.6289	1179.8	17¼	5.7393	2576	27	11.239	5044
3½	0.5245	235.40	10½	2.7256	1223.2	17½	5.8645	2632	27½	11.552	5185
3¾	0.5817	261.07	10¾	2.8234	1267.1	17¾	5.9908	2689	28	11.869	5227
4	0.6409	287.63	11	2.9225	1311.6	18	6.1176	2746	28½	12.189	5470
4¼	0.7020	315.06	11¼	3.0228	1356.6	18¼	6.2453	2803	29	12.510	5614
4½	0.7647	343.20	11½	3.1240	1402.1	18½	6.3738	2861	29½	12.835	5760
4¾	0.8292	372.14	11¾	3.2267	1448.1	18¾	6.5039	2919	30	13.163	5907
5	0.8956	401.95	12	3.3300	1494.5	19	6.6344	2978	30½	13.491	6055
5¼	0.9636	432.46	12¼	3.4344	1541.4	19¼	6.7660	3037	31	13.827	6205
5½	1.0333	463.75	12½	3.5403	1588.9	19½	6.8980	3096	31½	14.162	6356
5¾	1.1046	495.74	12¾	3.6470	1636.8	19¾	7.0309	3155	32	14.501	6508
6	1.1773	528.37	13	3.7548	1685.2	20	7.1650	3216	32½	14.842	6661
6¼	1.2523	562.03	13¼	3.8638	1734.1	20¼	7.2997	3276	33	15.186	6815
6½	1.3275	595.78	13½	3.9735	1783.3	20½	7.4354	3337	33½	15.533	6970
6¾	1.4048	630.47	13¾	4.0842	1833.0	20¾	7.5720	3398	34	15.881	7127
7	1.4836	665.84	14	4.1963	1883.3	21	7.7091	3460	34½	16.233	7285
7¼	1.5638	701.83	14¼	4.3092	1934.0	21¼	7.8469	3522	35	16.587	7444
7½	1.6454	738.46	14½	4.4230	1985.0	21½	7.9860	3584	35½	16.944	7604
7¾	1.7282	775.62	14¾	4.5382	2036.7	22	8.2662	3710	36	17.303	7766

* Warning: This table should be used only for rough approximations, as for spillway capacities. Velocity of approach is not considered.

Table 288. Triangular Weirs. Coefficient of Contraction

2A	Variation of head, ft.	Readings, number	Range of C	C* Average
28°	0.4775–3.0375	17	0.586–0.607	0.5994
60°	0.2303–2.9922	24	0.435–0.587	0.5645
90°	0.440 –2.458	15	0.532–0.591	0.5705
120°	0.294 –1.501	13	0.574–0.603	0.5953

* If one ignores heads below 0.5 ft., C = 0.6002, 0.5752, 0.5752 and 0.5966 for 28°, 60°, 90°, and 120°, respectively.

Proportional Flow Weirs. Weirs of the Sutro type are designed so that head above crest is directly proportional to the discharge. The proper shape of the curve of the sides has been investigated by Rettger and others; see *E. N.*, Jan. 25, 1914, p. 1409.

Trapezoidal Weirs. Cippoletti's formula is $Q = 3.367LH$, and applies to a weir with end slopes 4 on 1, sill horizontal, all edges sharp; the plane of the weir should not be more than 4° from a perpendicular to the axis of the canal; other conditions are the same as for Francis weir with full contractions. For lengths 3, 4, 5, 6, 7, 8, and 9 ft. of Cippoletti weir, Flinn and Dyer in 1893,[59] found an agreement with the standard Francis weir, within 0.2 to 5 per cent. for heads from 0.2 to 1.4 ft. The experiments were made at the Holyoke testing flume.

J. C. Stevens experimented on 6-in., 2-ft., and 3-ft. weirs. The discharge curves gave continually larger results than indicated by the Cippoletti formula.

These differences result from two causes: (1) On long weirs, the effect of end contraction, a curvilinear rather than a rectilinear function, is negligible, whereas it is a larger item on short weirs; end slopes should be greater than 4 on 1 properly to compensate for contraction. (2) The slightly rounded crest

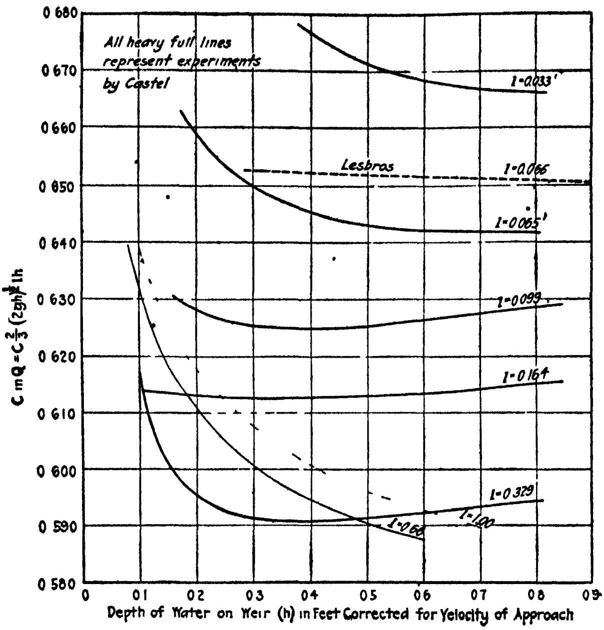


FIG. 333.—Coefficient C for short sharp-crested rectangular weirs, with end contractions.[58]
(Hamilton Smith, Jr)

reduces the bottom contraction. The weir tested had 4 on 1 end slopes. It was made of 2-in. plank cut to shape, and beveled 45° on the downstream side. A galvanized iron plate in one piece protected the edge.[60]

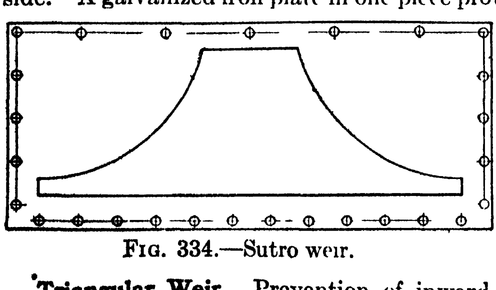


FIG. 334.—Sutro weir.

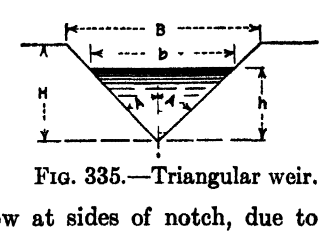


FIG. 335.—Triangular weir.

Triangular Weir. Prevention of inward flow at sides of notch, due to narrowness of channel or roughness of upstream face of weir, increases discharge.[61]

Angle at vertex 90°. $Q_m = C_w H^{\frac{5}{2}}$, in which C_w has following values:

h	0 25	0 333	0.417
C_w*	153.06	151.98	151.2

Q = cfs, Q_m = cu. ft. per min., h = head on notch, ft.

* Thomson's experiments[62] yielded, respectively, 153 3, 152 76, and 152 34

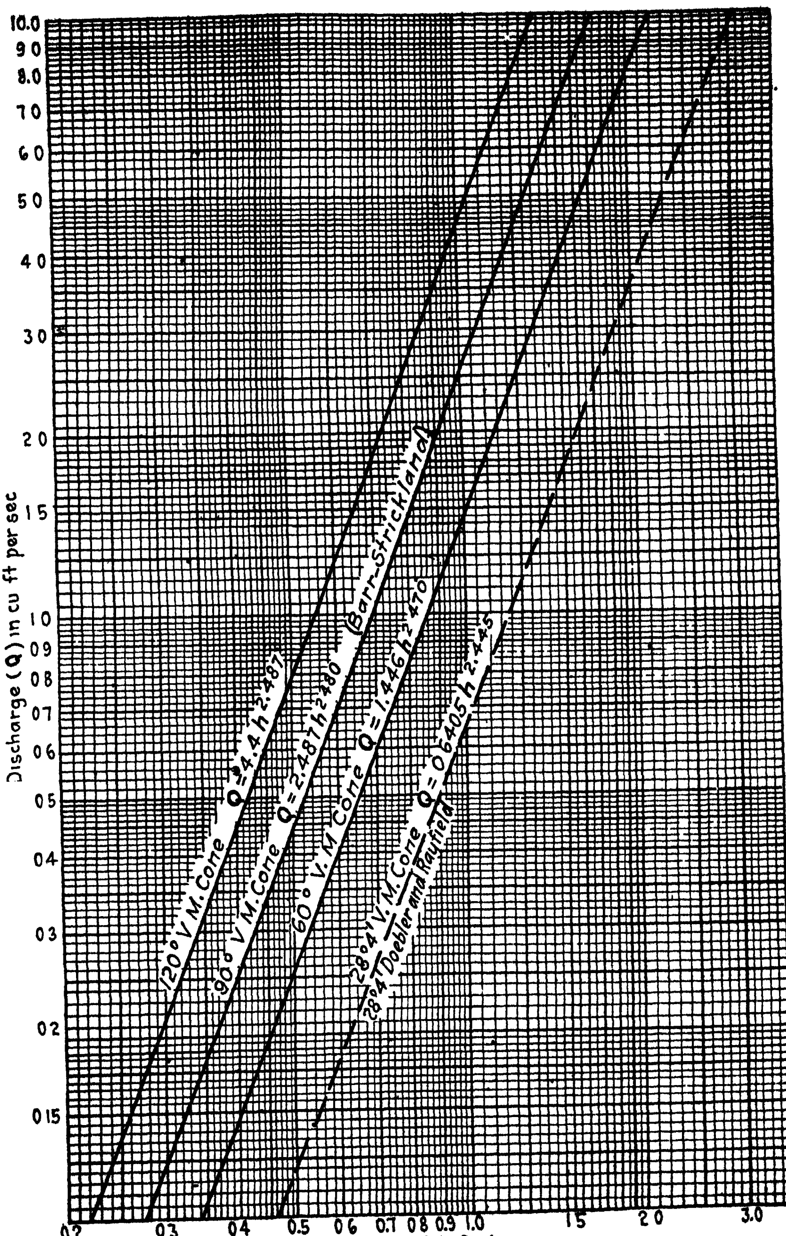


FIG. 336.—Discharge of triangular weirs.[64]
(Studies by V. M. Cone,* Doebler and Rayfield, Barr-Strickland)

* See also tests by Cone on circular notches, J Agri Research, U S Dept, Agri., Mar. 6, 1916; also E. N R, July 31, 1924, p 182

Angle at vertex 54°; area of orifice is half that for 90°; C_w is slightly greater than half C_w for 90°.

Tests at Cornell University: For perfect contraction, with sharp edges,

$$Q = \frac{4}{15}C\frac{B}{H}\sqrt{2g}.h^{\frac{5}{2}} = \frac{8}{15}C \tan A\sqrt{2g}.h^{\frac{5}{2}}$$

B = width of notch at height H. $A = \frac{1}{2}$ angle.

If $K = \frac{4}{15}\frac{B}{H}\sqrt{2g}$, a constant for a given weir, then $Q = Kh^{\frac{5}{2}}$ *theoretically*, or $Q = CKh^{\frac{5}{2}}$, where C = coefficient of contraction.*

From these data, experimenter R. B. Daudt concludes: (1) For heads between 0.5 and 3 ft., coefficient of contraction is fairly steady. Average C might be within 1 per cent. of truth. (2) Knifelike edge of weir secured almost perfect contraction, accounting for lower values of C than in other experiments. (3) Intermediate weirs between 28° and 120° have lower coefficients than these limits.[63]

Table 289. Discharges over a 90° Triangular Notch Weir

$Q = 2.64h^{\frac{5}{2}}$ (Q in cfs.; H in ft.)

h, inches	Gallons per min	Liters per min.	h, inches	Gallons per min	Liters per min.
1/4	0.076	0.29	3 1/4	45.1	171
1/2	0.42	1 6	3/8	49.7	188
5/8	0.74	2.8	1/2	54.4	206
3/4	1.16	4.4	5/8	59.2	224
7/8	1.71	6.5	3/4	64.6	245
1	2.38	9 0	7/8	70.1	266
1/8	3.19	12 1	4	75.9	288
1/4	4.17	15.8	1/8	82.1	311
3/8	5.25	19 9	1/4	88.3	335
1/2	6.55	24.8	3/8	95.1	360
5/8	7.99	30.3	1/2	102.0	387
3/4	9.60	36.4	5/8	109 2	414
7/8	11.4	43 0	3/4	116.7	442
2	13.4	51.0	7/8	124.6	472
1/8	15 7	60.0	5	132.7	503
1/4	18 0	68 0	1/8	141.1	535
3/8	20 6	78.0	1/4	150.0	569
1/2	23 5	89 0	3/8	159.0	603
5/8	26.5	100.0	1/2	168.5	639
3/4	30 1	114.0	5/8	178 3	676
7/8	33.4	127.0	3/4	188.3	714
3	37.0	140.0	7/8	198 8	753
1/8	41.0	155.0	6	209.5	794

Weir box is used to measure the discharge of small pumps, and other relatively small quantities. The depth on the weir must not be over one-third the depth of the box or canal, and the distance of the side of the weir opening from the side of the canal or box must not be less than the depth on the weir. A series of racks baffle flow effectively.[65] The edge over which water falls must be level. All edges of the notch must be sharp.

Flow of Water over Dams. From Cornell experiments on flow over weirs, Gardner S. Williams has deduced coefficients in Table 290 to be applied to Francis or Bazin weir formula (based on sharp-edged weir) in computations for discharge over dams.

* Values in Table 288, p. 787.

From measurements of flow the following rating formulas have been determined.[66]

$Q = 2.491\ Lh^{1.77}$	Merced dam
$Q = 3.088\ Lh^{1.50}$	La Grange dam *(Turlock)
$Q = 3.110\ Lh^{1.65}$	Yakima dam
$Q = 3.196\ Lh^{1.59}$	Old Austin dam (Texas)†
$Q = 3.043\ Lh^{1.484}$	Rock-faced dam (Birmingham)[48]

Table 290. Discharge Coefficients for Overflow Dams[67]

Type	*A*	*A*	*A*	*A*	*A*	*A*	*A*	*A*	*B*	*B*	*C*	*D*	*E*	*F*
b(feet)	0.48	0.93	1.65	3.17	5.89	8.98	12.24	16.30	6.65	11.25	.....	.....	.....	.....
Head (ft.)														
0.5	0.902	0.830	0.819	0.797	0.785	0.783	0.783	0.783	1.060	1.060	0.968	0.971	0.971	0.971
1.0	0.972	0.904	0.879	0.812	0.800	0.798	0.795	0.792	1.079	1.079	1.008	1.040	1.040	0.983
1.5	1.000	0.957	0.910	0.821	0.807	0.803	0.802	0.797	1.091	1.092	1.032	1.083	1.092	1.022
2.0	1.000	0.989	0.925	0.821	0.805	0.800	0.798	0.795	1.086	1.097	1.041	1.105	1.126	1.040
2.5	1.000	1.000	0.932	0.816	0.800	0.795	0.792	0.789	1.076	1.096	1.043	1.118	1.146	1.057
3.0	1.000	1.000	0.938	0.813	0.796	0.791	0.787	0.784	1.067	1.095	1.044	1.128	1.163	1.072
3.5	1.000	1.000	0.942	0.810	0.793	0.787	0.783	0.780	1.060	1.094	1.045	1.136	1.177	1.085
4.0	1.000	1.000	0.947	0.808	0.790	0.783	0.780	0.777	1.054	1.093	1.046	1.144	1.190	1.097

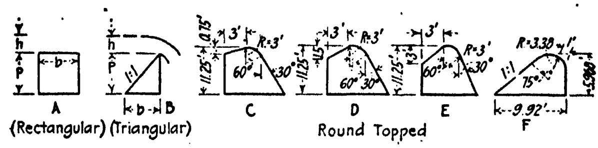


FIG. 337.—Types of crests referred to in Table 290.

Coefficients of Discharge for Flow through Notches.[68] In constructing dams, it is often necessary to approximate the number of openings to be left to take the river flow. Tests were made at McCall's Ferry dam, with rectangular openings 43 ft. deep, and 40 ft. wide, left in the concrete, 7 ft. above the river bed; each notch, or opening, was about 40 ft. long (=thickness of dam). Two formulas were tested for constants: (1) Fteley and Stearns:

$$Q = C_oL(H + \tfrac{1}{2}H_t)(H - H_t)^{\frac{1}{2}}$$

(2) Ordinary weir formula:

$$Q = C_wL(H - H_t)^{\frac{3}{2}}$$

Q = discharge, cfs.
H = measured height, ft., of upstream pond level over bottom of notch, corrected for velocity of approach.
H_t = measured height, ft., of tail water below dam over bottom of notch, corrected for velocity of retreat.
L = crest length, ft., corrected for end contraction = $40 - 0.2\,(H - H_t)$.
C_o = coefficient of discharge for submerged weir.
C_w = Bazin's coefficient, using head, $H - H_t$.

H varied from 14.2 to 24.4 ft., with a corresponding variation in H_t from 4.40 to 7.60 ft., and Q from 4550 sec. ft. to 10,100 sec. ft. C_o varied from 2.35 for first condition, to 2.47 for $H = 17$ and 18 ft. C_w varied from 3.92 to 4.01 corresponding to heads and quantities first mentioned.

* Profile on p. 173.
† Profile on p. 168.

MEASUREMENT OF FLOW*

Flow in *pipe lines* can be gaged by the insertion of a *Venturi meter* (see p. 459), an orifice diaphragm, or a pitot tube, when each is properly calibrated. Piezometers on a pipe line indicate by the difference in gage heights, the loss of head in the intervening pipe, whence can be derived the value of S in the Chezy formula, see p. 754, for calculating flows. For examples of use on Detroit tests, see *T. A. S. C. E.*, Vol. 47, 1902, p. 71. Salts and chemicals have been used with success in hydro-electric work. See B. F. Groat, in *T. A. S. C. E.*, Vol. 80, 1916, p. 951, and Allen and Taylor, *Mech. Eng.*, Vol. 46, 1924, p. 13. Open-channel flow may be gaged by the Venturi flume, weirs, or various irrigation devices. See bibliography in *Proc. A. S. C. E.* March, 1925, p. 154.

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* **For Streams,** see p. 39.

† **47 and 48** at end of list.

CHAPTER 35

MASONRY AND PUDDLE

CONCRETE*

Sand† for mortar or concrete in hydraulic structures may be coarse, medium, or fine, but must not have grains of uniform size nor a large proportion of substantially one size; a gradation from coarse to fine‡ which has minimum voids is best. Run of crusher or bank is used only by permission. The fineness modulus§ should, in general, be not less than 2.5 nor more than 3.5. Grains should be hard, durable material, with not more than 3 per cent. organic matter or 5 per cent. clay. In particular, sand should be free of organic coating. Field test for this consists in shaking sand with dilute solution of sodium hydroxide, setting aside for 24 hr., and noting the color of supernatant liquid.[41] Dirty sand should be washed; too uniform sand should be mixed with other. Some sands apparently clean will not make good mortar, some ingredients interfering with the setting of the cement; less than 1 per cent. foreign matter is sometimes troublesome. Sand which cannot be used with one brand of cement will sometimes give satisfactory results with another brand. Some mortars and concretes which harden very slowly ultimately become very strong. Best determination is to make large test cylinders or cubes of concrete or mortar with the sand and cement which it is expected to use. For cylinders 8 in. in diam., 16-in. length is standard, but 2 by 4 in. is commonly used.

Table 291. Standards for Classification of Gravels and Sands by Sizes*

Conventional name	Corresponding diam. mm.	Corresponding diam. in.
Coarse gravel	50 0 to 5 0	2.00 to 0.20
Fine gravel*	5 0 to 1.0	0 20 to 0.04
Coarse sand*	1.0 to 0.5	0 04 to 0.02
Medium sand*	0 5 to 0.25	0.02 to 0.01
Fine sand*	0.25 to 0.10	0.01 to 0 004
Superfine sand (very fine)*	0 10 to 0.05	0.004 to 0.002
Rock flour—silt*	0.05 to 0.01	0.002 to 0.0004
Superfine flour—silt*	0.01 to 0.005	0.0004 to 0 0002
Clay*	0.005 to 0.0001	...

* Designations by U. S. Bureau of Soils, *Bull* 249, 1912.

Cement.‖ Only Portland cement should be used for important structures. Its cost on works has been reduced by (*a*) manufacturing from native rock

* See the various standard books, notably the section on Materials in "Concrete Engineers' Handbook," by Hool and Johnson (McGraw-Hill Book Company, Inc , 1918).

† See *E. R.*, June 12, 19, 26, 1915, articles by Chapman and Johnson

‡ New York Central R. R. practice classifies as sand all material passing a No. 3 sieve (¼-in. mesh).

§ The fineness modulus is the sum of the percentages in the sieve analysis divided by 100, when the sieve analysis is expressed as percentages coarser than the sieves in the Tyler standard screen scale; ranging from 100-mesh screen as the finest, and with no upper scale.

‖ See Specification C9–21, A. S. T. M.

at the site, as was done at Roosevelt and O'Shaughnessy dams, or by grinding locally with equal parts of sand to form "sand cement," as at Elephant Butte and Arrowrock dams.[2] The latter sets more slowly than cement, and requires larger investment in forms; the tendency to cracking also increases.[2]

Tufa Cement. Sometimes tufa is a calcareous deposit, but that described here is of volcanic origin. It is of grayish or creamy color, and of low specific gravity in rock form. As tufa is light and easily crushed, the grinding machinery is cheap. Tests and field experience in massive structures on the Los Angeles aqueduct show the tufa cement to be as strong and as satisfactory generally as straight cement when used in concrete, and very much cheaper, as tufa deposits were found along the line. Generally, it was used in equal volumes with Portland cement, mixed by regrinding together.[3] Laboratory tests over 5 years showed continued increase in strength.

Alcement (alumina cement)[4] hardens more quickly than Portland cement, and has very high early strength, so that outlay for forms is reduced, but particularly the time of putting work into service is accelerated. The time of set is about the same as for Portland, but great strength is immediately developed. It is being used at intersections of important highways and similar places where it is desirable to put the construction in use the morning following placing, and for repairing Portland cement concrete floors and roads.[5] It has been developed in Europe to be fully resistant to sea water or sulfate attack. It costs two to three times as much as Portland cement and is as yet limited in production. It cannot be mixed with Portland cement; it must be maintained absolutely wet during the setting period—4 to 12 hr. after placing; to avoid shrinkage and development of excessive heat it should not be used richer for the mortar part of concrete than 1 cement to 2 sand; and the adjoining work and forms should be thoroughly wetted at placing. Portland cements are being studied to produce a high early-strength product and are now being produced on a limited scale in Switzerland (Holderbank "Speciale").

Solubility of Cement. Experiments at the laboratory of the Catskill aqueduct, N. Y., showed the Portland cement in lean concrete through which fresh (Croton) water had percolated for nearly 2 years was dissolved to such extent that the strength of the concrete was only 20 to 35 per cent. of normal for concretes of the given age and mixture. The sand was standard crushed quartz and the coarse aggregates included glass at one extreme of a series of specimens and a soluble limestone at the other. The aggregates were unaffected and seemed to have had no effect on the result. Specimens through which the greatest quantity of water had passed were weakest. The proportions of the concretes were $1:3\frac{1}{2}:6$. The pressure tunnels of the Catskill aqueduct, on the contrary, showed no effects of solubility of cement, but became tighter year by year (Stebbins).

Concrete specifications are standardized by the work of the Joint Committee,* and should be enforced. Under unavoidable circumstances only should this practice be departed from. Sand used at Glen Lake dam[7] was not of the desired quality, but satisfactory concrete was produced. Bank-

* See *Proc. A. S. C. E.*, October, 1924, p. 1154; also Specifications C33–23T (Concrete Aggregates), C40–22 (Test for Organic Impurities in Sands for Concrete), and C44–22T (Rules for Inspection of Concrete and Reinforced Concrete Work), A. S. T. M.

run gravel screened down to 6 in. was used on Stevenson dam at a saving of $250,000.[7] Concreting at Bassano reinforced-concrete dam was not stopped until temperature reached −15°F.; water and gravel were heated, and all forms were double-boarded.[6] A culvert on Canadian Pacific Ry. was built successfully when temperature ranged from +20 to −50°F.; see report of Committee on Placing Concrete in Winter, reported to Am. Bridge & Bldg. Assn.[8] In cold-weather work, steam pipes under canvas have a tendency to dry by evaporation and thus injure the concrete; supplement with or use dead steam, as the setting concrete will not then be robbed of necessary moisture (Stebbins).

Proportioning Concrete.* The *Ideal curve* of Taylor and Thompson[9a] as a guide for concrete on hydraulic work is limited in application because: (1) the percentage of cement has often been fixed by specification irrespective of the character of the aggregates (it should not be); (2) there are few places where more than one source of sand and stone is available, allowing small choice in aggregates; (3) grading material by screening the stone into several sizes is generally impracticable. The proportion of sand determined by the *ideal curve* must be increased to get a plastic mix. Where watertightness is of primary importance, proportion the concrete so that the mortar is in excess of the voids in the large aggregate by 15 to 20 per cent. (sufficient to make the mixture plastic and to compensate for irregular mixing and placing). The density test is the most scientific method of determining proportions. Make up trial mixtures of the materials to be used, according to the consistence intended. The materials are carefully weighed and mixed, and a batch placed in an 8-in. wrought-iron pipe 10 in. long capped at one end, and the height of concrete noted. Several batches should be tried in which the weights of cement and water, and the total weight of aggregates remain constant, but the proportion of sand to stone is varied. That proportion is densest which gives the smallest volume of concrete. Use the proportion nearest this, which at the same time looks well and works smoothly. Measurements on construction work show a tendency to run up the proportion of sand, as this facilitates mixing and placing.

Abrams' methods are outlined in *Bulls.* 1 and 3, Lewis Institute Structural Materials Laboratory, 1925.† On the Becks Run bridge, Pennsylvania R. R., tests undertaken in skepticism gave gratifying results. Proportioning on the basis of the fineness modulus, and fixing water content by slump test, led to nearly exact predictions of strength of test cylinders. On Newark Bay bridge, Jersey Central R. R., the Abrams' method was reported "efficient, easy of application, and sure in results." A determination of proportions could be made within an hour after receipt of aggregates.[10] Time alone will indicate the quality of work secured. Concrete for Black Canyon dam,[11] U. S. Reclamation Service, was so proportioned "with excellent results." Stebbins insists that no method should be allowed to supersede judgment in the control of proportions, for the modulus (*i.e.*, proportions) suitable for road work would be too "mushy" for mass concrete, and too "rocky" for tunnel lining.

* See *E. R.*, Jan. 23, Feb. 6, 13 and 27, and Mar. 6 and 13, 1915, articles by N. C. Johnson.
† "Design of Concrete Mixtures."

Slump and flow tests are used to measure the consistence or workability of concrete.[12] In field tests reported to A. S. C. E.,[10] a truncated cone was used, 4 in. in diam. at top, 8 in. at bottom, and 12 in. high. The cone is filled with freshly mixed concrete and immediately withdrawn. The slump is the settlement, in in., of the mass of concrete from its original level at the top of the cone. (Consult D62–20T, Vol. 21, Pt. 1, 1921, A. S. T. M., for criterions.) The slump at Camden work ranged from 4 to 10 in., with a 1:2:4 mix, while at Newark Bay, with a mix of 1:2.4:3.6, it ranged from 0 to 8 in., and averaged 3.5.

The flow test requires special apparatus, known as a flow table. This test when made in accordance with the recommendations of U. S. Bureau of Standards, consists of jigging a cone of concrete on a flow table and measuring the increase of bottom diam. of the concrete. The concrete is molded in a truncated metal cone with a top diam. of 6.66 in., bottom diam. of 10 in., and height of 5 in. The form is withdrawn immediately after molding. The flow table is raised $\frac{1}{8}$ in. and dropped by a cam, 15 times in 10 sec. The flow is the ratio of the increase of base diam. to the original diam. Table 292 indicates the ranges in field tests for Joint Committee on Standard Specifications.*

Table 292

Item	Camden tests				Newark Bay tests			
	1:2:4		1:1:2		1:2.4:3.6		1:1:3	
	Slump	Flow	Slump	Flow	Slump	Flow	Slump	Flow
Average	7.5	85	8.4	94	3.5	22	3.9	20
Maximum range	4.0–10.0	20–150	6.8–10.0	50–138	0.0–8.0	2.5–60	1.5–6.0	10–30
Range for 80 per cent. of tests (that is, extremes eliminated)	6.0– 8.5	50–120	7.5– 9.0	65–115	1.5–6.0	5–42.5	2.0–6.0	12.5–27.5
Maximum range of daily averages	5.7– 9.1	38–125	7.2–10.0	65–120	0–6.2	3.8–40.8	2.5–6.0	10–27.5

Mixing by weight was recommended by Johnson[13] to secure a strong, impermeable concrete for oil reservoir at Three Rivers, Que., as the local sand varied considerably in volume with moisture content. A $\frac{1}{2}$-cu. yd. batch consisted of 3 bags cement, sand weighing 5.04 bags and stone weighing 10.85 bags of cement—substantially a 1:1.67:3 mix. Water ratio was 0.6; the mixture worked well. The composition and the character of the concrete were thus made dependable, and forms could safely be pulled earlier than usual.

U. S. Reclamation Service specified for American Falls dam† (1924): Only sufficient water shall be used to secure a workable mix sufficiently fluid to flow properly into place with thorough spading and working and for the heavy part of the work no softer consistency than 3-in. slump will be permitted, while the reinforced and thin work may go up to 6 or 8 in. of slump. Concrete is to be mixed at least $1\frac{1}{2}$ min.

Water. Tests by Abrams[40] on different kinds of contaminated water—alkaline, sewage polluted, etc.—gave unexpectedly good results. Neither odor nor color indicates suitability of water. Use of common salt to lower freezing point of mixing water during winter should not be permitted; 5 per cent. salt lowers freezing point 6°F., but reduces strength 30 per cent. Increasing the quantity of mixing water reduces strength of concrete.

* See *Proc. A. S. C. E.*, Oct., 1924 and Jan., 1925.
† See *E. N. R.*, Nov. 6, 1924, p. 742.

To secure good concrete experience on the Catskill aqueduct points to the following requirements:

(*a*) Any concrete can be greatly damaged and many times ruined by drying during the rapid setting period; sprinkle with water continuously from initial set through the first night and the day following, and then maintain in a moist condition for several days; dead steam under canvas completely supersaturating the air for 3 days is effective. (*b*) Use plenty of cement; for waterproof concrete in cut-and-cover aqueduct not less than 1.4 bbl. (5.6 bags) were used per cu. yd.; in pressure tunnel lining, 1.7 (6.8 bags) to 2.0 (8.0 bags); in fence posts 2.0 (8 bags). There is a tendency to limit the cement in the interests of "economy." For durability where exposed to atmosphere use not less than 7 bags per cu. yd. and pay particular attention to the curing. (*c*) Mix thoroughly. (*d*) Do not allow separation during transportation, during placing, or after placing. (*e*) Thoroughly spade the concrete both at the forms and around all reinforcing to force out all air. (*f*) Do not use an excess of mixing water, or allow the concrete to become "mushy" in the forms. (*g*) For fine aggregate a mixture of natural sand and crusher grits from hard rock was found superior to either alone.

Durability of Concrete.* In 1848 the Erie R. R. built Starrucca viaduct, using over 1000 cu. yd. of 1: 1.5: 3.5 Rosendale cement concrete. F. L. Stuart, Chief Engineer, stated that an examination in 1907 showed the concrete in good preservation. The Department of Docks and Ferries, New York City, has built many miles of Portland cement sea wall, some of it previous to 1875; it is in good condition, except for about 2.5 ft. near low-water mark, where the waves and ice have eroded it slightly. In 1907 D. K. Colburn, Bridge Engineer of Galveston, Harrisburg & San Antonio Railway Co., had occasion to remove the tops of concrete piers built in Medina River in 1881; the concrete was found in perfect condition. It was composed of a 1:1.66 mix of Portland cement and irregular flinty pebbles. For exhaustive studies, see report on "Marine Structures," to National Research Council, by Atwood and Johnson, 1924.

Alkali† has caused deterioration of concrete in irrigation and other structures. Alkali in ground water forced drainage measures for Winnipeg aqueduct.[14] Rapidity of deterioration appears to depend on sulfate content of water, according to investigations by Bureau of Standards. Portland cement as now constituted is inherently susceptible to attack by sulfates in solution.[41] On construction of headworks for Lethbridge Northern Irrigation District, mixing water was tested with a Jackson turbidometer, and no water containing more than 150 p.p.m. of sulfates was permitted to be used.[43]

Erosion of Concrete by Water.‡ Observations show: (1) that concrete is not injured by clear water gliding over it at high velocities (up to 40 f.p.s.) if velocity or direction is not abruptly changed; (2) that concrete subjected to the impact of water at high velocities is rapidly eroded.[16] Vacuum has been known to break down concrete in blow-offs, and in the culvert at Cal-

* See "Chemical Resistance of Engineering Materials," by M L Hamlin and F M Turner (Chemical Catalog Co , 1923), and Baylis, "Corrosion of Concrete," *Proc A S C. E.*, Apr., 1926, p. 549–579. See also Chap 17, p 375.

† See also p 375

‡ See also *E N R.*, Jan 24, 1918, p 173; and "High-pressure Reservoir Outlets," U. S. Reclamation Service, 1923.

averas dam, where concrete of high quality was disintegrated and eaten away to a serious extent.[17] A water-wheel nozzle, lined by William Mulholland, Los Angeles, with Portland cement mortar to reduce the diam., showed all the marks of the wooden mandril about which the mortar was placed, after several years' use. Tests by E. C. Clark[18] indicated that 1:2 Portland cement mortar is the best proportion for resisting abrasion, the resistance being nearly double a 1:1 or 1:2.5 mixture; for natural cements, 1:1 gave the best results. The South Canal, Uncompahgre Valley project,* has carried up to 900 sec. ft., at a velocity of 25 to 30 f.p.s.; it is operated at varying heads during each irrigation season. No erosion has taken place in 14 years, and no extensive repairs have been required. At power plant, Strawberry Valley project,* Utah, velocities of 30 to 40 f.p.s. have been resisted for 16 years by a reinforced-concrete spillway on a slope varying from 18 to 42 per cent. and carrying from 30 to 125 sec. ft.

The San Fernando open conduit of the Los Angeles aqueduct was designed for a maximum velocity of 22.3 f.p.s.[19] The blow-off of San Maria Lake dam[20] under 56-ft. head from two 48-in. pipes tore a lining of ¼-in. steel plate into many small pieces.

Turlock dam, see p. 192, is submerged by water laden with sand, with a drop of about 100 ft.; 18 years' wear, under velocities as high as 70 f.p.s., has had slight effect on the concrete at the base; erosion has been greatest on the sand and cement, leaving the greenstone aggregate projecting in spots; ½ in. is the maximum wear. The wear can probably all be laid to the sand in the water. A turbulent stream discharging under 60-ft. head at Roosevelt dam, through an unlined tunnel, did considerable damage. After discharging water for 6 months through the Pathfinder dam at velocities of 75 to 90 f.p.s., marks of the forms still showed plainly on the concrete lining; a turbulent stream through a conduit of nearly double this cross-section, under the same head, injured the lining. Concrete sewer inverts in Duluth, Minn., showed no deterioration after 20 years, while brick sewers, built at the same time, had to be renewed in 6 or 7 years. One drain, 4 ft. in diam., 2000 ft. long, on a 13 per cent. grade (velocity 42 ft. per sec.), had an invert of flat granite flags, laid with 1:1 Portland cement mortar. There was a heavy storm flow, carrying sand, etc. After 2 years' wear, the ridges at the joints indicated that the mortar was more durable than the granite.[9b]

Tar coating for concrete surfaces[21] to prevent erosion by water at high velocities was used in the regulating outlets of Arrowrock dam by Construction Engineer, C. H. Paul. Diam. of outlets, 4 ft. 4 in.; surface, 1:2 Portland cement mortar; velocities of water, 60 ft. To fill all minute voids, surfaces were scraped and washed with grout and then painted two coats of water-gas tar and two coats of coal tar. It is important that the water-gas tar be of very thin consistence; it can be so obtained if so ordered of the manufacturers, who can also supply an oil for thinning, if necessary; success depends upon having this tar of a water-like consistence. Both water-gas and coal tars should be refined; they may be obtained from Barrett Mfg. Co. Concrete should be thoroughly dry when first coat is applied, but water gas may be applied without heating; the second coat may follow the first immediately. Both

* Correspondence with Bureau of Reclamation, U. S. Dept. of Interior.

coats of coal tar should be applied hot and brushed out as thin as possible; first may be applied as soon as the water-gas tar has soaked in a little, but second should not be put on until first has set. A thick coating is likely to peel and run. After a year's trial, these coatings were found to have been thoroughly satisfactory. The cost was about $1\frac{3}{4}$ cts. per sq. ft. (1915). For 1000 sq. ft. there were required: labor, $4\frac{1}{2}$ days at $2.50; 12 gal. water-gas tar and 15 gal. coal tar, at 16 cts.; brushes and miscellaneous, $1.43. Price of tar was $4 per 50-gal. barrel plus freight $4.

Waterproofing Concrete.* *Permeability tests*† and studies of concrete were made at laboratory of Catskill aqueduct, New York. Permeability is subject to accidental variations in a much greater degree than strength; yet a study reveals certain governing laws. The quantity of cement used is important. Concrete of properly graded natural aggregates containing over 15 per cent. of cement by weight was found to be practically impermeable under heads not exceeding 200 ft. As the percentage of cement was reduced, permeability increased slowly until about 11 per cent. was reached, when the permeability increased rapidly. Proper grading of the aggregates is another important condition. High percentages of fine particles produce a decided reduction in permeability, while tests with natural sand, of which 25 per cent. was finer than 0.01 in. and 10 per cent. finer than 0.006 in. in diam., in a 1:3 mortar, resulted in practical impermeability. When made from broken stone and screenings, concrete is more permeable than when made from gravel and natural sand. Sand gave more uniformly dense concrete than crushed stone screenings. Concrete is more permeable in a direction parallel to its bed than perpendicular thereto.

Tests on lean concrete mixes for drainage blocks showed that no concrete of sufficient strength to be handled in blocks can be depended upon to be permeable enough to act as a free drain. Horizontal stratum of relatively impermeable concrete is formed at the bottom of the block mold, which should be chipped off to obtain the greatest permeability.

Surface Skin. The dense skin formed by screeding or by casting against metal was found by Board of Water Supply tests to have an important effect on permeability.‡ These tests indicated permeability under water pressure of 1:2.6:4.9 concrete, 6 in. thick, with the skin intact, to be only 1.2 per cent. of that of the same concrete without surface skin; as a resistant, the skin is equal to many feet of concrete. With richer concrete this proportion would be reduced; nevertheless watertightness depends largely on the continuity of the skin. Surface skin produced by troweling concrete as it sets is a great help toward waterproofing. Pittsburgh filtered-water reservoir floor has two layers of concrete each 4 in. thick, in blocks lapping 3 in., each surface being troweled hard. This reservoir holds 25 ft. of water without measurable leakage, although the floor rests directly upon loose, sandy gravel, extremely pervious. Rich surface coatings of 1:1 or 1:2 mortar are often applied to horizontal or inclined surfaces. Concrete should be surfaced while green and the coating troweled in place to produce a thickness of $\frac{1}{2}$ to 1 in. Heat

* Much of the following on Waterproofing is from "A Treatise on Concrete, Plain and Reinforced," Taylor and Thompson (Wiley, 1916).
† See "Waterproofing Engineering," by Ross, Wiley, 1919, p. 220.
‡ See Moore, *J. Assn. Eng. Soc.*, September, 1911, pp. 111, 112.

causes crazing. Such coatings may be good for submerged surfaces of reservoirs. Plastering on concrete, however, is rarely successful unless done by specially skilled workmen with faithful attention to numerous details. Tests by University of California of cement-gun coat 1 in. thick on Gem Lake dam under pressures of 700- to 1600-ft. head for 4.25 and 25 hr., respectively, showed neither leakage nor moisture.[22]

Rich Concrete. Permeability may be decreased by proper mixing. The richest concrete up to a certain point shows the least permeability; mixes richer than 1:3 are liable to crack. Permeable concrete tends to become tight after the passage of water through it. The mix should be such as to secure dense concrete with excess of mortar. Thorough mixing produces a homogeneous mass. Wet mixes are tighter than dry, if not excessively wet. Satisfactory results have been secured from 1:3 to 1:6 mixtures. Prolonged and thorough mixing is important, but even more important is scrupulous care in placing so as to avoid all porous or honeycombed spots. Another very important detail is to keep the concrete thoroughly wet from the time it begins to harden until it is about 2 weeks old, and to protect it from the hot sun. Concrete should not merely be lightly sprinkled occasionally, but thoroughly wet, and not permitted to dry out for about the period named, depending upon temperature, humidity, thickness of mass, and other local conditions. Tests by Crook and Faulkner[23] led them to conclude that impermeability depends upon the curing, regardless of richness of mix or grading of aggregates. Moisture must be conserved to assure the hydration or crystallization of the cement, which cannot proceed when moisture is lacking, and which produces crystals, which, in expanding, fill the voids, promoting impermeability, and, in interlocking, give increased strength. Experiments at National Physical Laboratory, England,[34] indicated to Johnstone-Taylor that impermeability depends not so much upon richness of mix as upon proper proportioning.

Sylvester's "Process for Repelling Moisture from External Walls" has been used some time in England; it was used in United States in 1870 on the Central Park gate house.[24]

On Strawberry Valley project, vertical surfaces tending to scale were subjected to Sylvester process, and horizontal surfaces were brushed with paraffin, which was forced into the pores by flashing the flames of a blowtorch over the surface.[28]

Concrete Paints.* Linseed oil paints should never be applied directly to either new or old concrete. In order to neutralize the lime, 1 lb. of sulfate of zinc should be dissolved in a gallon of hot water and sprayed or brushed on concrete and then allowed to dry for a few days. After that a priming coat of any good paint containing a little spar varnish may be applied.

A much better method is to use an acid resin paint as a primer, which neutralizes the lime directly into an insoluble compound, over which a good linseed oil paint may be applied or a good spar varnish paint.

Great care must, however, be taken that only alkali-proof pigments are used, so that, in case there are any spots where lime may come through, the pigment is not destroyed.

* Maximilian Toch.

Incorporated Waterproofing. Board of Water Supply* studies were made of the effects of various materials incorporated in the matrix of the concrete in reducing permeability, including clays, hydrated lime, and puzzolan and sand cements, these two cements being more finely ground than Portland. The conclusion was reached that concrete, practically impervious under a head of 200 ft., could be produced with any of these materials, but that equally good results can be secured at no greater cost by using sufficient Portland cement, the latter method having the advantage of increasing the strength, while other materials usually reduce it. I. O. Baker[31] says that a safe practice is to dissolve both the alum and the soap in the mixing water, 1 part alum to 2 parts hard soap by weight—any reasonably pure soap. As not more than 3 per cent. hard soap can be dissolved in cold water, the alum is limited to 1.5 per cent. of water weight. Professor Baker also gives the following formula in his "Masonry Construction:" 1 per cent. by weight of alum is added to dry cement and sand; 1 per cent. potash soap or ordinary soft soap is dissolved in the mixing water. An insoluble compound results. W. K. Hatt has successfully used 5 per cent. solution of alum and 7 per cent. solution of soap, in equal parts in mixing concrete. The use of 3 lb. of hydrated lime to a bag of cement gave good results on the Baltimore & Ohio R. R. Medusa[32] requires 6 lb. per bbl. of cement. Many patented compounds are on the market, claiming that, by addition of 1 to 2 per cent. by weight to cement, permeability is reduced Many of these, by test, have proved useless or worse; chemical analyses show some ingredients to be impermanent and others injurious to concrete. Intelligent selection of aggregates, skilful, thorough mixing, careful placing, and subsequent protection from wind and sun, and keeping wet for about 2 weeks are the best means for making concrete watertight.

In general, the use of waterproofing ingredients or applications induces carelessness, as contractor will rely on them to cover slovenly work. It is held by many far better to put the value of the waterproofing materials into the concrete itself by adding more cement and securing better workmanship.[33]

Membranes.† There are many types, the purpose of all being to furnish a cover completely surrounding the structure to be waterproofed. They are impregnated felt, laid in several overlapping layers, fastened to the masonry by bituminous compounds. Success depends largely upon the skill with which they are applied; continuity of membrane is essential; as the bituminous membrane is likely to be punctured, it must be protected on the exposed side by a lining, generally of brick. Many have been successfully applied to repair leaky concrete standpipes (see p. 535).

Concrete Surfaces Protected from Chemicals. At the Minneapolis filters,[35] inside surfaces of chemical-solution tanks are kept well covered with good grade of asphalt or graphite paint. Alum solution was decomposing the concrete by attacking the limestone aggregate. Graphite paint seems to give better service than asphalt. A new paint, "Frost's Kapak Special Acid-resisting Paint," appears to give as good results as graphite; it is somewhat cheaper. Inertol‡ is used on many Imhoff tanks, on the surfaces exposed to sewage.

* New York City.

† Firms supplying membranes include the Barrett Co., Briggs Bitumen Co., Wailes-Dove Corp., all of New York.

‡ Karl Fezer, New York.

CYCLOPEAN MASONRY

To procure good work, following conditions are essential: Concrete should be mixed very wet and should always be dumped in a low spot on the dam or wall so that the water is confined and cannot drain out. If racking is used, small rubble walls should be built to confine the concrete. Fine aggregate for concrete should not be composed entirely of rock screenings, as this does not make a well-graded mix, and forms a concrete which is "bony," making it hard to bed the large stones; such concrete does not retain its fluidity. A suitable quantity of well-graded natural sand should be added to the screenings. Concrete should be mixed in batches and should be placed as soon after mixing as possible, otherwise it settles in the bucket, and is hard to dump. Batches larger than 2 yd. are hard to work over and spread. Care should be taken to avoid laying stones with concave beds or long flat projections, such stones being hard to settle into the concrete, and likely to entrain air and water. In Burren Jock dam, Australia, bond of cyclopean masonry was increased by a system of units, cruciform in shape, with area of 1080 sq. ft. These units vary in height from 9 to 15 ft., being so arranged that continuous units break joint both horizontally and vertically[36] (see also p. 157 for other construction experience). List on pp. 161–167 includes some cyclopean masonry dams.

Weight of Masonry in Dams. Church and Fteley, in their report on Quaker Bridge dam, assume weight of granite masonry at 156.25 lb. per cu. ft.; the Board of Experts on this dam assumed 146.25, citing Krantz as using 143.7, and Bouvier, 147.3. Records from Boyd's Corners dam gave 146.0 lb.; 1:3:6 concrete will weigh about 154 lb.; 1:3 mortar, 140 lb. If the masonry contains 30 to 40 per cent. granite "plums," 55 to 65 per cent. of 1:3:6 concrete, and 5 per cent. mortar, it weighs about 157 to 158 lb.; if 30 to 40 per cent. limestone "plums," about 149 to 150 lb., depending on the specific gravities of the aggregates in the concrete and of "plums" or large stones, and on density of the masonry. Rubble in New Croton dam weighs 156.5

Table 293. Cyclopean Masonry
Proportion of Large Stones

Name of dam	Percentage of large stone	Cement, barrels per cu yd of masonry
Boonton	50	....
New Croton (south end)	50	....
Cross River	34.5	0.754
McCall's Ferry	18	....
Cataract	20	....
Shoshone	25	....
Roosevelt	51.3	0.71
Pathfinder	48.5	....
Burraga	33	....
Barossa	14	....
Vyrnwy	54	....
Granite Springs	65	....
Ashokan	30	0.90
Gilboa (1:1.9:5.3)	7+	1.088
Kensico (1:3·6)	27.1	0.837

lb., and in Sweetwater, 164.0. Cyclopean in Vyrnwy dam weighs 155.0; in Boonton, 166.0; and in Sand River, 150.0 lb. In calculations for Olive Bridge dam, cyclopean was taken at 145.5 lb., and in Cataract[37] dam, 140.

GROUT*

For grouting dam foundations, tunnels, shafts, and other places, especially where cold water under pressure is encountered, a quick-setting cement is often necessary. Grout sets much more slowly than mortar. Portland cements endure this severe use better than natural cements because the latter cannot stand the larger proportion of water. Natural-cement grout has been found of a putty-like consistence after several months. Grouting was employed extensively on Catskill aqueduct (see summary of experience by Sanborn and Zipser, *T. A. S. C. E.*, Vol. 83, 1920, pp. 980–1080; and articles by Sanborn, *E. R.*, Apr. 15, 1916 *et seq.*). Caniff grouting machines are made by Ransome Concrete Machinery Co., Dunellen, N. J.

PUDDLE†

Puddle is a mixture of clay, sand, and gravel, moistened and thoroughly compacted into certain parts of hydraulic structures to prevent seepage or percolation of water. Proportions differ with the characteristics of the ingredients and the details of use. In some places nearly, or quite satisfactory, natural mixtures are found; in some instances, mixing is done crudely in the trench, in others a pug mill or similar machinery is employed and as much care taken as with concrete. It is used as cores in earth dams, on the bottoms and slopes of reservoirs where they are pervious, beneath bottoms and on outsides of walls of filters and masonry reservoirs. Ingredients should be carefully selected and mixture determined by trial. Stones large enough to interfere with thorough mixing and compacting must be removed. Use only water enough to insure moistening throughout the mass. Clay should not be in so large proportion as to make the puddle too slimy or cause shrinkage and cracks in drying. Thoroughness of compacting has much to do with prevention of cracks. Puddle should not be allowed to dry out; if work suffers long interruption, puddle should be sprinkled occasionally with water or covered with canvas, boards, moist earth, or other protection. In finished structures, puddle should be protected by earth, masonry, or stone paving, so that it will not be eroded by water, dried out, or exposed to frost. For the Philadelphia filter plants, puddle was made of 1 part plastic red clay from Swedeland, Pa., 1 part red clay from Delaware City, Del., and 2 parts sandy gravel or broken stone smaller than ¾ in. in greatest diam. Clay was ground in pug mills to reduce lumps and thoroughly mixed with stone or gravel; water was added to make stiff paste. While plastic, it was spread in an 8-in. layer on a prepared earth bottom, in large areas, and while drying repeatedly rolled with grooved rollers weighing ½ ton per lin. ft. of roller, until all shrinkage cracks were closed; thickness was reduced to 6 in. and two more layers added, making total thickness, 12 in. Such puddle was very impervious and strong. The concrete of the filter bottoms was placed on it.

* See also p. 126.
† See also p. 195.

Prof. Carl Hilgard[38] found that the addition of 0.1 per cent. carbonate of soda will make almost any earth available for puddling. When puddle paving is used, if sprinkled with saltpeter, KNO_3, before rolling, it will not crack nor wash off.

Fanning gives the following formula for an especially dense, strong puddle, which will successfully resist water, rodents, and eels:

In practice, 7 measured cartloads of coarse gravel mixed with 3 of fine made about 8 loads, which were spread in 2-in. loose layers in the trench; on the gravel, 2 loads of clay were spread, and lumps broken, and on top 1 load of sand. Clay loads were slightly smaller than the others. This triple layer, thoroughly mixed with a harrow, was then moistened to a kneading consistence and thoroughly compacted into a solid mass by a 2-ton grooved or ring roller. Such puddle may cost as much as concrete (a larger bulk being used) and in most cases would not be as satisfactory.

Table 294. Gravel-clay Puddle; Proportions of Ingredients

Materials	Per cent voids	Cu. yds. required	
		Theoretically	Practically
Screened coarse gravel	28–30	1 00	1 00
Fine gravel	30	0 28	0 35
Sand.......	33	0 08	0 15
Clay .		0 03	0 20
Total materials		1 39	1 70
Resulting puddle		1 00	1 30

Bibliography, Chapter 35. Masonry and Puddle

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CHAPTER 36

METALS*

BRONZES

Sluice Gate and Valve Seats.† Bronzes *A* and *B* were used in the sluice gates of the Pathfinder and Shoshone dams, U. S. Reclamation Service (O. H. Ensign); *C* and *D* were used for Catskill aqueduct. These bronzes have the following approximate compositions in percentages:

Metal	Class			
	A(a)	*B*(a)	*C*(b)	*D*(b)
Copper	83.5	81.8	82.8	82.7
Lead	8.1	4.9	8.0	4.9
Zinc	4.5	5.3+[1]	4.4	5.3
Tin	4.9	7.1+[1]	4.8	7.1
Total	101.0	99.1	100.0	100.0

(*a*) by analyses of test pieces; (*b*) by specification. To test bronzes *A* and *B*, pieces 1 in. wide and 2 in. long were moved backward and forward on pieces of opposite composition, 1 in. wide, with 6-in. stroke, under a load of 3200 lb. per sq. in., at planer speeds; no squealing, chattering, nor dust developed after several hundred strokes. Catskill aqueduct specifications allowed a variation in per cent. of any one ingredient not exceeding 0.75, and required: A piece of *C* bronze, at least 16 in. long and 2 in. wide, finished on one side, shall be firmly secured, and a piece of *D* bronze, 8 in. long, 2 in. wide, similarly finished, rubbed back and forth along its surface for at least 500 strokes at speed of at least 35 ft. per min., the two metals being forced together with a pressure of not less than 1000 lb. per sq. in., and the surfaces lubricated with water only; neither metal shall be abraded nor cut, nor shall there be any chattering nor screaming. Specimens from one melt of each bronze having satisfactorily passed this abrasion test and chemical analysis, following melts were passed on analysis and behavior during machining. *C* and *D* bronzes have been successfully made by several manufacturers. Tests were usually made in a planer.

Friction of bronze on bronze[1] under high pressures was studied by U. S. Reclamation Service. These bronzes have about the composition given in column A, above.

The gates tested had been operated previous to the experiment at intervals of not over 2 months.

* See also Chaps. 15 and 18 and the index.
† See also pp. 436 and 454.

Table 295. Friction Tests, Bronze on Bronze[1]

Item	Pathfinder	Arrowrock	Elephant Butte
Size of gates (larger dimension vertical), feet	4.42 × 7.5	5 × 5	3.92 × 5
Head on center of gates, feet	127	48.5 to 54.8	107.3
Average time of opening, minutes	16	10	5.2
Average time of closing, minutes	14	11	4.2
Friction coefficient:			
Average for opening stroke	0.42	0.31	0.336
Maximum for opening stroke	0.44	0.42	0.383
Average for closing stroke	0.29	0.29	0.254
Maximum for closing stroke	0.31	0.34	0.268

Manganese bronze[2a] is a brass, the manganese being simply a deoxidizing agent; the alloy often contains but traces. Its strength, ductility, and resistance to water give it a wide field of usefulness; it has replaced aluminum bronze in many uses.[1] Trouble has been encountered with rolled and extruded manganese bronze due to cracks from high internal stress. Bronze bolts, subject to fatigue failure, if overstressed in assembling, should not be used in vital places.[3] Board of Water Supply specifications provide:

All forged, rolled, or extruded bronze shall be subject to test with a scleroscope, and if a hardness is found materially exceeding that typical of hot-worked metal, the bronze shall be rejected or annealed promptly, as directed.

The strength requirements are:

	Minimum requirements		
	Castings	Forgings (*a*)	Forgings (*b*)
Ultimate strength, pounds per square inch	65,000	70,000	80,000
Yield point, pounds per square inch	32,000	35,000	40,000
Elongation, per cent	25.0	28.0	22.5

Provision (*b*) was adopted in 1922 to secure a stiffer stem and one less likely to grind in the nut.[3] Welding or burning for patching imperfect castings is prohibited; there are few if any burned castings which have not cracked; some cracks have not developed for years.[3] Chemical properties are not specified in recent contracts. See also specification B17-14, A. S. T. M.

Cracking of Brass and Bronze. Pipes, tubes, bars, bolts, rods, plates, and wires of common brass, manganese bronze, and naval brass sometimes crack where not under external stress and for no *apparent* reason. The cause, so far as now determined, commonly is: too severe cold drawing in finishing, improper heat treatment, or lack of annealing, or wrong methods of final fabrication. Such cracking rarely develops within a month of manufacture, and may not appear for several months or even years. These defects should be guarded against for important uses. In the present state of the art, the only safeguard seems to be insistence on the products of reliable manufacturers, and having all work, especially such as involves any heating or deforming of the metal, done only by persons experienced and skilled in the particular kind of work to be done. This defect can be detected by methods described

in B21-19, A. S. T. M. Exposure to extreme cold, as during a winter, expedites the cracking. Manganese bronze, naval brass, common, and other brasses must be used with caution, in rolled, drawn, and similar forms. Castings of these alloys, so far as known, are subject simply to the usual mishaps of the founders' art, and are useful for many engineering purposes.*

Failure of brass service pipes at Pittsburgh is laid to electrolytic action, caused by a mixture of copper, lead, and zinc which is not complete and homogeneous.[4] Tobin-bronze filter plates in Minneapolis failed to such an extent as to require replacement with Monel metal.[5]

Monel metal† is a natural nickel-copper alloy produced from ores of the Cobalt, Ont., district. Its three important mechanical properties are incorrodibility, toughness, and retention of strength at high temperatures. It is used in valve parts for high-pressure fire service in Boston,[6] for pump rods and liners by U. S. Navy, for chlorine machinery, etc. It has a coefficient of expansion of 0.00001375 per 1°C.; modulus of elasticity, 23,000,000 to 24,000,000; yield point, casting specimens, tension, 37,000; ultimate tensile strength, 72,000. It is more difficult to melt, cast, and machine than the brasses and bronzes.

Gaskets for Flange Joints. For bronze flanges under great pressure where durability was essential, soft lead wire gaskets were adopted on Catskill aqueduct. Table 296 gives results of some tests. "Enduro" and "Velumoid" paper gaskets have also been used successfully.

Table 296. Lead and Copper Wire Gaskets: Compression Tests

Material	Initial diam. in.	Length, in.		Load, lbs.		Final thickness, in.	Remarks	
		Initial	Final	Initial	Final			
Lead	0.387	5.0●	....	2,310	10,150	0.250	Down flat	Each load (5 to 8 loads) on 1 min.
Lead	0.500	5.0	....	2,970	26,600	0.253	Down flat	
Lead	0.382	5.0	....	2,300	10,000	0.253	Down flat	
Lead	0.383	2.47	2.52	1,000	14,000	0.253	Down flat	
Lead	0.499	2.50	2.67	1,500	12,000	0.252	Down flat	
Annealed Copper	0.311	5.01	....	3,490	70,000	0.253		
Annealed Copper	0.311	5.0	....	8,000	64,000	0.250		

Corrosion. Bronzes are but slowly affected by ordinary conditions of earth or atmosphere. Evidence is afforded by numerous objects collected among the ruins of ancient cities in various parts of the world, many having inscriptions still legible, although they have been exposed to air and moisture for more than 20 centuries. A great quantity of prehistoric bronze axes, knives, etc., in good preservation, have been taken from lakes in southeastern France. Modern bronzes have apparently the same freedom from corrosion under ordinary conditions. Manganese, Tobin, and phosphor bronzes are little affected by sea water and have proved satisfactory where brass and steel have proved useless. Bronze in the gates of the Old Croton aqueduct, in service since 1842, so far has been unaffected. No trouble has been expe-

* For many years, brass founders have repaired defects in brass and bronze castings by "burning in" or other process of welding Some recent experiences demonstrate that such repairs cannot be permitted on hollow castings to contain water under pressure. If made by a man of rare skill and long experience, the repairs may be successful, provided due regard is had for all the conditions in each case.

† Produced by International Nickel Co., New York.

rienced from corrosion of bronze in stopcocks or hydrants in Croton Waterworks, New York City.

Tests by Board of Water Supply.* Alloys embedded in moist earth 6 months all showed more or less corrosion. Embedded partially in concrete and submerged in Esopus Creek, rate of corrosion was ⅛ in. in about 1100 to 4600 years. Experiments at Buenos Aires[7] with city water which contains no unusual constituents and is slightly alkaline due to bicarbonates showed that typical brasses and bronzes immersed for 702 days suffered a loss of weight from 14 to 19 milligrams per sq. cm.

CORROSION OF METALS†

Copper oxidizes very slowly in the atmosphere, being covered with a film of carbonate, commonly called "verdigris," forming a protective coating, which, however, erodes or is slightly solvent in the weather.

Tin is unaffected by corrosion in the air, beyond the formation of a thin film on the surface.

Zinc is easily acted on by moist air; a film of oxide is soon formed, however, which protects the metal. If the moisture of the atmosphere contains acid, the zinc may be destroyed.

Lead.‡ The chemical properties of lead are peculiar and present some very remarkable contrasts. While it resists sulphuric and hydrochloric acids in a far higher degree than iron, zinc, or tin, it is readily attacked by weak organic acids, and is slowly dissolved even in pure water containing air. It is soon extensively corroded when exposed to moist air in the presence of carbonic acid. Lime mortar, lime putty, and lime water will attack lead; if the mortar is very alkaline, the effect will be greater. John Newman[8] reports an instance of lead pipe embedded in cement found deeply corroded; the action was destructive near the end of the pipe next to a basin, diminishing as the pipe receded from the water; the same authority gives several cases of damaging corrosion of lead pipe from contact with mortar or cement. A weighed sample of lead immersed 8 days in water seeping through concrete walls in a tunnel under sea water, at New York, was taken out daily, washed, dried, and weighed; progressive corrosion took place, amounting to 0.132 per cent., or at the rate of 6 per cent. per year. In a bath establishment, two lead pipes pass through a concrete floor, a layer of asphalt, and tile embedded in cement mortar; one pipe for hot and the other for cold water; the room temperature, practically constant throughout the year, was 80°F. Water, high in free carbonic acid, lime and magnesia, was freely used on that floor; the lead pipe for hot water was corroded completely off in 5 to 10 years; the cold-water pipe was deeply corroded; no probability of electrolytic action existed. In

* See Annual Reports, 1908, 1909.

† Literature on corrosion is extensive. National Research Council, Washington, has a cooperative committee studying the subject comprehensively, which has compiled a selected bibliography. See, "The Corrosion of Metals," by Ulric R Evans (Longmans Green, 1924); and "The Corrosion of Iron," by J. Newton Friend, Iron and Steel Institute, Carnegie Scholarship Memoirs, Vol. 11 (Spon, London and New York, 1922). The latter summarizes knowledge to date. The Library of Queen's University, Toronto, published a bibliography of 2000 references in 1923. The Mellon Institute of Pittsburgh has prepared a bibliography of the literature pertaining to corrosion. A committee of the Standardization Council of the A. W. W. A. has also reported on corrosion, and in April, 1925, the Division of Industrial and Engineering Chemistry of the American Chemical Society held a symposium on the subject; *Ind. Eng. Chem.*, Vol. 17, 1925, p. 335; also see National Research Council Reports. See also "Corrosion" by Speller (McGraw-Hill Book Company, Inc. 1926).

‡ For lead services, see p. 587.

1860–1861 a portion of St. Petersburg* was served by a system of lead mains; in 10 to 15 years these pipes had become so damaged by corrosion as to necessitate extensive changes; the trouble was attributed to a local peculiarity of soil, the exact nature of which was never ascertained. In ordinary atmospheric corrosion, lead is one of the most durable of the common metals, undergoing no change in dry air or in water perfectly free from air; it is only slightly affected by hard waters or dilute solutions of either hydrochloric or sulphuric acids; but is readily dissolved by water high in nitrates and by dilute nitric acid; waters which actively corrode lead are those with a slightly acid reaction, from peaty swamps; soft waters are particularly unsuited for conveyance in lead pipes. If exposed to clean, soft water containing the normal quantity of dissolved oxygen, the lead is oxidized to hydroxide, which dissolves; the waters which act least on lead are those which contain carbonate of lime, phosphate of lime, and, in a less degree, sulphate of lime. Newman quotes various experiments to show that lead water pipes should be kept full of water all the time to prevent deterioration. Since lead acts as a cumulative poison, its salts produce serious results if taken into the human system even in very minute quantities for a length of time.

Under whatever conditions lead withstands water or acids, bronzes are similarly unaffected, while under other common conditions in which bronzes are practically untouched, lead would be destroyed. Lead-lined cast-iron pipe has been destroyed by well-defined electrolysis, and in these cases the lead was eaten through as well as the iron (or steel). Lead affords doubtful protection to iron or steel water pipes and under some conditions might prove a disadvantage. The coefficient of expansion of lead is 2.5 that of steel; hence it is questionable, whether in large pipes or specials, lead lining would remain in absolute contact. The high specific gravity of the lead, coupled with its great expansion, confers a tendency to work downward if in a vertical or sloping position. If water finds its way between the steel (or iron) and the lead lining, lead being electropositive to steel, energetic galvanic action and rapid corrosion of the pipe metal would follow.

Tests at Purdue University[9] on self-corrosion of lead cable sheaths indicated that most important cause was organic matter in soils, producing acids on decomposition. Alkalies, limestone, concrete, gypsum, etc., accelerate corrosion. Order of decreasing corrosiveness was found to be muck, cinders, sand, and clay. Tin-lead alloy appears more resistant than pure lead. Corrosion may be prevented by enclosing lead pipe in a clay duct. Use of concrete or untreated wood in contact with the lead should be avoided.

Corrosion of iron and steel is the subject of various debatable theories (see Marks, "Mechanical Engineers' Handbook" (McGraw-Hill Book Company, Inc., 1923), p. 587, and footnote on p. 808.) Bureau of Standards[10] is studying corrosion of underground pipes. Studies of 900 specimens in 1924 indicate that initial corrosion of iron and steel is rapid in certain soils in the South and Southwest.

Dissolved oxygen is held largely responsible for the corrosion of con iron and steel by ordinary water. This gas becomes more active temperature rises. Corrosion will be reduced by removing the

* Petrograd, now Leningrad.

oxygen and preventing its reentry. *Deactivation* may be accomplished by chemically fixing the oxygen by contact with a mass of iron or steel, with about 65 sq. ft. of surface per cu. ft. of tank space,[2b] or by driving off the entrained and dissolved air by heating, or by a combination of the two methods. The Kestner "degasser" is based on the first principle, water passing through a two-compartment chamber, where the flow may be diverted to either chamber or reversed in direction, to afford means of cleaning the corrosion from the expander. Cheap metals used in the degasser save about five times their weight of more expensive metal in the pipes, etc.[11] The "degasser" is commonly used prior to boilers, and in connection with the Kestner continuous feed treatment and blow-down system. See "Control of Corrosion by Deactivation of Water," by Speller, *J. Franklin Inst.*, April, 1922; and "Corrosion" by Speller (McGraw-Hill Book Company, Inc., 1926).

Alkalinity. In natural waters, over a fairly wide range, variations in hydrogen-ion concentration have no effect on rate of corrosion. Total acidity may be more important than actual pH value[12] (contradicted by Speller *J. A. W. W. A.*, Vol. 12, 1924, p. 419. *Ind. Eng. Chem.*, Vol. 16, April, 1924, p. 393). Studies by Speller and Texter[13] on corrosive effect of alkaline solutions on steel pipe indicated that, after a protective coating of ferric hydroxide is formed, alkalinity has an inhibiting effect, and that this film is a larger inhibitive factor than the decreased hydrogen-ion concentration. The protective film will not last long when alkali-free water is again run through the pipes.

Experiments at Louisville[14] indicate that removal of suspended matter by filtration, without use of coagulants, accelerates corrosion 100 per cent., showing that river silt acts as a preservative of iron when the water contains free carbonic acid and oxygen in solution. It is supposed that this preservative action is effected by mixing of suspended particles with ferric hydrate of iron, thus shielding the surface of the metal where corrosion is taking place.

Cast iron rusts slowly in air or in fresh water, but is rapidly corroded in salt water (see p. 418), in which it gradually becomes soft.* Mallet[15] found that the rate of corrosion decreased with the thickness of the casting, being from 0.1 to 0.4 in. in depth during a century for castings 1 in. thick. Under some conditions, cast irons differ greatly in resistance to corrosion. Foundry "skin" of natural surface is highly resistant compared with machined surface. Old cast-iron pipes, taken up or cut into from time to time, are found in good condition in many localities; old pipe thus removed is often in good enough condition to be relaid. Gas companies in New York have had practically no trouble from the corrosion of cast-iron pipe except to a limited extent along the river front. All mains are now of cast iron. Experience with wrought-iron and more particularly with steel pipe has been unsatisfactory on account of corrosion. Much depends on local conditions, coating, carefulness in construction, etc. At Rochester, N. Y., no rust leaks had occurred in a cast-iron conduit 14½ mi. long after 32 years' service. During the same period in a wrought-iron conduit 13 mi. long, there had been seven rust leaks, whereas in a parallel steel conduit 29 mi. long, given the best protective coatings then

* See "Graphitic Corrosion of Cast Iron," by J. Vipond Davies, Publ. 6, Engineering Foundation, 1926, p. 36.

known, passing through the same kind of soil and thus subjected practically to the same conditions, there were 164 rust leaks in 16 years, or one-half the period of service of the cast-iron conduit.

Experience seems to show that cast iron resists corrosion better than either wrought iron or steel. Cavallier, director of foundries at Pont-a-Mousson, France, mentions an intake pipe at Paris, extending into the river Seine, laid in 1802. Taken up after more than a century of immersion, it was found to be in fine condition. The same authority reports that the fountains at Versailles are supplied through cast-iron pipes laid in 1685.[16] For external corrosion, see *T. A. S. C. E.*, Vol. 78, 1915, p. 806.

Cast-iron pipe laid in Philadelphia[17] in 1827 was still in serviceable condition when removed in 1915. Analysis of incrustations: iron oxide (Fe_2O_3), 77.40; sand (insoluble matter, SiO_2), 7.85; moisture, 2.16; volatile matter (organic), 11.95; total, 99.36 per cent.

Tropics.[18] Many years of observation in tropical climates show that iron and steel, when not in contact with the earth, remain remarkably free from corrosion. This is especially so in the Amazon Valley. On the Madeira-Mamore R. R. rails (probably iron), manufactured in Belgium in the early sixties of the past century, as well as machinery and locomotives, were remarkably free from corrosion after more than 40 years even where overgrown with vines and brush. The rails still showed furnace scale on the web. Some rails that were lying on the rock bottom of the river, submerged half of the year, and exposed to the air the other half, showed very little corrosion.

Ingot iron.[19] Corrosion tests by Am. Ry. Eng. Assn. showed it substantially as corrosive as charcoal iron or Carnegie open-hearth steel, when subjected to corrosive influences of four test mediums: sand, alkali soil, clay soil, and cinders.

Wrought-iron bolts and iron work in foundations of Third Ave. bridge, New York, put in before 1860 were taken out in 1894 in perfect condition, grease being still on the threads, continuously submerged in salt water. Nuts were easily started with wrench (J. B. Goldsborough). Wrought-iron bearing piles submerged 23 years, in brackish water, exposed to sea air, Ozama River bridge,[20] Santo Domingo, were "hardly damaged" when removed from the bridge.

Corrosion of Steel Pipe.* Electrolysis occurs as a natural process in buried iron or steel pipes inadequately protected, if the ground conditions afford media for solution and electrolytic conduction (see also p. 428). The presence of minute amounts of soluble salts in wet soils may lead to destructive electrochemical action. In the Rochester pipe line, soil conditions played a leading part in corrosion. The damage was confined to very wet soils. A portion of the Portland, Ore., steel pipe laid in trench entirely in sand in the river bottom was found in excellent condition; in wet clay the pipe was much pitted and rusted. The Atlantic City 30-in. steel main illustrates the fact that earth and water of salt marshes are actively corrosive to buried metallic structures; the soil was very acidulous. When clay soils are impreg with salt or brackish water, they are much more corrosive than sandy similarly situated. At New Bedford, Mass., a steel pipe passes thr 2 mi. of peaty swamp. Gravel was hauled in to use as pipe covering;

* See also p. 764.

has been no trouble from corrosion. Peat is considered non-corrosive by some authorities. Peat may be corrosive or non-corrosive, depending upon its acidity or alkalinity. At Cambridge, Mass., the greatest damage was done where the soil was a moist sandy clay. Experience seems to show that both wrought iron and cast iron resist corrosion better than does steel.

Authorities differ as to relative corrosion of wrought iron and steel in house vents and drains. Investigation in Chicago[21] in 1918 of installations in service 20 years disclosed wrought iron 20 per cent. destroyed, cast iron, 25 per cent., and steel, 90 per cent. Similar results* were obtained by Gerhard[22] from study of vents on 98 buildings in New York City. But Speller and Walker,[37]† from studies at Boston, conclude that well-coated steel pipe is more durable than wrought iron. At Bangor, Pa.,[23] 30-in. steel pipe, $\frac{3}{16}$ in. thick, exposed to ice and floods, lasted 22 years.

Steel dam at Ash Fork, Ariz.,[24] was well preserved after 27 years.‡ Original coating was two applications of dry red lead and boiled linseed oil. Repainted twice in 18 years, each time with one coat of Dixon graphite. "Aquatite" applied in 1923.‡

Composition of water will affect rate of corrosion. Steel-needle nozzles in hydroelectric plant of Aomori Electric Co.,[25] Japan, were eroded to a depth of $\frac{3}{4}$ to $1\frac{1}{2}$ in. behind the entire circumference of the bronze insert, due to chemical composition of water, which contained 100 mg. of SO_3 per L. Bronze casting substituted. Steel is eroded by high-velocity water. Water screened to contain less than 0.1 per cent. solid matter, extensively eroded in 18 months nickel-steel needles, valve seats, and buckets in power house 2, Southern California Edison Co.[26] Flow was about 80 cfs. under 1960-ft. head.

Preventing Corrosion of Iron and Steel.§ *Mortar and Concrete.* Concrete is not an electric insulator; a small fraction of an ampere of electricity will corrode steel embedded in concrete and disintegrate the concrete. In sea water concrete offers less resistance than in fresh.[27] Conclusions from 50 tests of electrolysis by Institute of Industrial Research, Washington,[31] on painted steel rods in 1:2 Portland cement mortar.

Corrosion of metal embedded in concrete structures, by stray currents of high voltage, is often productive of serious effects. Use of properly made paints upon such metal constitutes a safeguard that should not be neglected. Such paints may be prepared from the following substances: The vehicle should contain: (1) boiled or bodied oils or products which dry to a fairly saturated film; (2) oils which dry by semipolymerization rather than oxidation; (3) oils which dry to a flat rather than a highly glossy surface. The solid portion should contain a percentage of: (1) pigments which are coarse and which, therefore, tend to form films having a rough surface; (2) pigments which are inert and which do not act as conductors of electricity; (3) pigments which are either basic or of the chromate type. The painted metal should be "sanded" if possible.

An examination of Bryn Mawr siphon, Catskill aqueduct, after 7 years, disclosed complete protection of the steel by the concrete lining.[32]

* Investigation sponsored by the wrought-iron interests.
† Investigation sponsored by the steel interests.
‡ Data from Atchison, Topeka and Santa Fé Ry.
§ See "Marine Structures," by W. G. Atwood and A. A. Johnson (National Research Council,

Iron embedded in fresh mortar rusts rapidly;* in pure cement it remains unrusted, even when kept under water.[28] A piece of iron in a sandstone masonry pedestal about 200 years old was badly rusted. A piece of cast iron from the Tower subway shell, London, cut out after 16 years, was in good condition. It had been coated with coal tar, with an outer coat of hydrated lime.[29] In a steel vessel, after 10 years' use without care, it was found that the unprotected floor in the boiler space was completely gone; under the boilers, where there had been a coating of practically pure Portland cement, the shell and angle bars were found in absolutely perfect condition.[30] A standpipe and tower at Louisville, after 21 years were in a badly rusted condition; they were cleaned and coated inside with 1:1 Portland cement mortar. Cleaning and coating were done in 8 hr., water being pumped the other 16. After some trouble, pipe was finally coated in a satisfactory manner.[33] C. Hermany says that this standpipe blew down in March, 1890; cement lining was in perfect condition and rust action completely stopped. Cement paint is largely used by railways in France on bridges, the iron being brushed, dampened, and given two coats of cement and sand. Dr. Goslich states that iron spirit tanks, painted inside with Portland cement, are universally employed in European distilleries.[34] L. L. Buck argues that limestone aggregate should not be used in concrete which is to come into contact with steel, as a soluble stone in contact with the steel may be reached by the water, and a cavity ⅛ to ¼ in. deep formed.[35]

Treatment of Water.† In incrustations of water mains at Colombo, Ceylon,[36] 1.5 to 8.0 p.p.m. iron (ferrous and ferric), sulphur, and iron bacteria as *Leptothrix ochrea* are present. Chlorine up to 5 p.p.m. and lime up to 4 g.p.g. killed bacteria but only treatment by coke filters, sedimentation, and slow sand filters stopped incrustations.

Galvanizing. *Specification*, North Jersey District Water Supply Comm. After being thoroughly cleaned and dried, articles shall be acceptably coated with zinc, which shall uniformly cover all parts so as to weigh not less than 1⅛ oz. per superficial sq. ft. All faces shall be measured in determining the superficial area. Coating shall be free of buckles, blisters, or other defects. All articles shall be fabricated as completely as possible before galvanizing. Except where permitted in special cases, hot process or Sherardizing shall be used. Galvanizing shall be of such a quality as to endure at least four consecutive immersions of 1 min. each in a solution of copper sulphate crystals having a sp. gr. of 1.185 at 70°F. For method of determining weight of coating, see Specification 90–23T, A. S. T. M.

A steel signal tower at Iloilo,[38] P. I., exposed to seashore influences sufficient to cause ⅛-in. scale on reinforcing bars exposed less than 1 year, and where paint applied in three coats had to be renewed in 1 to 2 years, galvanized after members had been cut and punched, showed no rust after 12 years. Galvanized bolts were used instead of rivets. High first cost.

Gerhard[22] found on roof vents that galvanizing afforded less protection to steel pipe than to wrought-iron on account of the smoother surface of the former.

* Not borne out by other observations.
† See also pp. 661 and 670.

Coatings (see p. 815). *Painting Steel Work.* A good paint for the first coat consists of 20 lb. of red lead and 1 lb. of lampblack in about 3 qt. boiled linseed oil, the red lead and lampblack being ground in. This will suffice to cover 50 sq. yd. For the other coats some durable oil or asphaltum paint should be used. For painting iron or steel work a good formula is: one coat red lead in oil, one coat elastic paint, one coat or more of desired color containing zinc oxide (40 per cent.), lead oxide (50 per cent.), and asbestos filler (10 per cent.). When repainting, all metal surfaces should first be carefully cleaned by a sand blast or by steel brushes and scrapers (good scrapers can be made by drawing the temper from old files, and sharpening like chisels). Rust spots should be removed even if considerable time be required, for if such spots are covered by paint, corrosion will still continue underneath and, as the rust swells, it will blister and push off the paint. Paint will harden better if applied when the metal is warm, and in any event the metal should be thoroughly dry. A steam hose is useful for drying steel work.

If the steel is coated with a paint which dries to a flat and fairly saturated film by semipolymerization, the pigments of which are either basic or of a chromate type which is inert and a poor conductor, Gardner[31] reports that the corrosion need not be serious.

Tests by Paint Manfrs. Assn. of U. S., on 3 years exposure on metal, showed equal durability for brush and spray coats. Practically all paint and dip coatings tend to act as osmotic membranes and so, by the establishment of acid-alkali cells, tend not only to hasten corrosion but also to accelerate their own destruction.[32]

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CHAPTER 37

CAPACITY AND CONVERSION TABLES*

QUANTITY OF WATER

Gallon, New York Statute measure (1829) = 8 lb. of distilled water or 221.184 cu. in., with barometer at 30 in., and temperature, 39.83°F. The legal gallon of United States is the old British wine gallon, containing 231 cu. in., being the contents of a cylinder 7 in. diam. and 6 in. high; it contains 8.3389 lb. of water at maximum density, weighed in air at 30 in. mercury pressure, and 62°F. Also = 0.1337 cu. ft. = 0.8327 imperial gal.

Imperial gallon contains 10 imperial lb. of distilled water at 62°F. weighed in air of the same temperature, and with barometer at 30 in. 1 imperial gal. = 1.200 U. S. gal. = 277.27 cu. in. = 0.160 cu. ft.

1,000,000 gal. = 3.07 acre-ft.

1,000,000 cu. ft. = 22.95 acre-ft.

Hogsheads, Hurst's rule. Capacity in U. S. gal. = $0.0017l(P+m)^2$, where P = product of inside diam. at the heads, in.; for equal diam., d, $P = d^2$. m = inside diam., in., at the bung. l = length, in., between insides of heads.

1 ton of water (2000 lb.) = 240 gal. ±.

1 drop water, 4°C. or 39°F. = 1 ♏ (minim) (U. S.) = 0.0616 c.c. = 0.00376 cu. in. = 0.0000162 gal. Experiments by E. G. Bradbury[1] indicated that 16,000 drops of water from metallic surfaces are approximately equal to 1 gal., a somewhat larger volume per drop than given by Kuichling, who found 1 drop per sec. equal to 3 gal. per day.

RATES

1,000,000 gal. daily (mgd.) = 1.547 cu. ft. per sec. = 694.44 gal. per min.

1 drop (♏), or 1 minim, per sec. = 1.40 gal. per day.

1 sec.-ft. or cu. ft. per sec. (cfs.) = 448.8 gal. per min. = 1 in. (roughly) per acre per hr. = 646,272 gal. per 24 hr. = 38.4 Colorado miner's in. = 50 Utah or Nevada in. = 40 California (Law of Mar. 23, 1901) or Arizona in. = 1.98 acre-ft. in 24 hr. = 59.5 acre-ft. in 30 days = 724.5 acre-ft. in 1 yr. = 1 acre-ft. in 12 hr. 6 min. = 86,400 cu. ft. in 24 hr. = 0.6463 17 mgd.

1 sec.-ft. for 1 yr. = depth of 13.572 in. on 1 sq. mi.

1 in. of rainfall per month (30 days) = 0 . 8962 cfs. per sq. mi. = 0.5793 mgd. per sq. mi.

1 cu. ft. per sec. per sq. mi. = 1.115 in. of rainfall per month.

1 mgd. per sq. mi. = 1.726 in. rainfall in 30 days = 1.55 sec.-ft. per sq. mi.

1 mgd. per acre = 0.000000247 sec.-ft. per sq. in. = 990.22 sec. ft. per sq. mi.

1 in. on 1 acre = 27,154.3 gal. = 3630 cu. ft. = 0.0833 acre-ft.

1 acre-ft. = 43,560 cu. ft. = 325,850 gal.

* For more complete tables, see Hering's "Conversion Tables" (Wiley) and Trautwine's 'neers' Pocketbook" (The Trautwine Co.).

1 in. on 1 sq. mi. = 17.379 million gal. = 2,323,200 cu. ft. = 53.33 acre-ft. = 0.0737 sec-.ft. per year.

1 in. per hr. on 1 acre = 0.652 mgd. = 1.0086 cfs. = 2.0 acre-ft. per day.

1 in. per hr. on 1 sq. mi. = 417.15 mgd. = 645.333 cfs. = 1280 acre-ft. per day.

1 in. per month (30 days) on 1 sq. mi. = 0.8962 cfs. per sq. mi. = 0.579 mgd. per sq. mi.

1 in. per sq. mi. per day = 26.8889 cfs. (nearly 27).

Miner's inch is defined as the quantity of water that will pass through an orifice 1 sq. in. in cross-section under a head given in various localities as 4 to 6½ in. to the center of the orifice. In states where the miner's inch is defined by statute, it varies from $\frac{1}{40}$ to $\frac{1}{50}$ cu. ft. per sec. It is going out of use, being superseded by cu. ft. per sec. and gal. per min.

Table 297. Converting Discharge in Second-feet per Square Mile into Run-off in Depth in Inches, over the Area[2]

Discharge in sec.-ft. per sq. mi.	Run-off in inches				
	1 day	28 days	29 days	30 days	31 days
1	0.03719	1.041	1.079	1.116	1.153
2	0.07438	2.083	2.157	2.231	2.306
3	0.11157	3.124	3.236	3.347	3.459
4	0.14876	4.165	4.314	4.463	4.612
5	0.18595	5.207	5.393	5.578	5.764
6	0.22314	6.248	6.471	6.694	6.917
7	0.26033	7.289	7.550	7.810	8.070
8	0.29752	8.331	8.628	8.926	9.223
9	0.33471	9.372	9.707	10.041	10.376

Note. For partial month, multiply the values for 1 day by the number of days.

Table 298. Converting Discharge in Second-feet into Run-off in Acre-feet[2]

Discharge in sec.-ft.	Run-off in acre-ft.				
	1 day	28 days	29 days	30 days	31 days
1	1.983	55.54	57.52	59.50	61.49
2	3.967	111.1	115.0	119.0	123.0
3	5.950	166.6	172.6	178.5	184.5
4	7.934	222.1	230.1	238.0	246.0
5	9.917	277.7	287.6	297.5	307.4
6	11.90	333.2	345.1	357.0	368.9
7	13.88	388.8	402.6	416.5	430.4
8	15.87	444.3	460.2	476.0	491.9
9	17.85	499.8	517.7	535.5	553.4

Note. For partial month, multiply values for 1 day by the number of days.

VELOCITY

1 ft. per sec. = 0.682 mi. per hr.

1 mi. per hr. = 1.467 ft. per sec. = 88 ft. per min.

POWER*

1 hp. requires 8.80 sec.-ft. falling 1 ft. = 5.68 mgd. falling 1 ft. at 100 per cent. efficiency.

1 mgd. falling 1 ft. = 0.176 hp. at 100 per cent. efficiency.

* See also p. 472.

1⅓ hp. = about 1 kw.

1 cu. ft. per sec. falling 1 ft. = 0.114 hp.

To calculate waterpower quickly: $\frac{\text{sec.-ft.} \times \text{fall in ft.}}{11}$ = net horsepower on waterwheel realizing 80 per cent. of the theoretical power (see also p. 471).

For 95 per cent. efficiency, waterpower = $\frac{\text{sec.-ft.} \times \text{fall in ft.}}{9}$

PRESSURE

1 ft. head or depth of water = 0.433 lb. per sq. in. = 0.029 atmosphere.
1 lb. per sq. in. = 2.307 ft. of water = 0.068 atmosphere.
1 atmosphere = 33.90 ft. of water = 14.7 lb. per sq. in.
1 ft. water column = 0.883 in. mercury.
1 in. mercury column = 13.59 in. water.

Table 299. Converting Inches Vacuum into Feet Suction

In. vacuum	Ft. suction	In. vacuum	Ft. suction	In. vacuum	Ft. suction	In. vacuum	Ft. suction
¼	0.28	8¼	9.35	16¼	18.42	24¼	27.50
½	0.56	8½	9.64	½	18.71	½	27.78
¾	0.85	8¾	9.92	¾	18.99	¾	28.07
1	1.13	9	10.21	17	19.28	25	28.35
1¼	1.41	¼	10.49	¼	19.56	¼	28.63
1½	1.70	½	10.77	½	19.84	½	28.91
1¾	1.98	¾	11.06	¾	20.13	¾	29.20
2	2.27	10	11.34	18	20.41	26	29.48
2¼	2.55	¼	11.62	¼	20.70	¼	29.76
2½	2.84	½	11.90	½	20.98	½	30.05
2¾	3.12	¾	12.19	¾	21.27	¾	30.33
3	3.41	11	12.47	19	21.55	27	30.62
3¼	3.69	¼	12.75	¼	21.83	¼	30.90
3½	3.98	½	13.04	½	22.11	½	31.19
3¾	4.26	¾	13.32	¾	22.40	¾	31.47
4	4.54	12	13.61	20	22.68	28	31.75
4¼	4.82	¼	13.89	¼	22.96	¼	32.03
4½	5.11	½	14.18	½	23.24	½	32.32
4¾	5.39	¾	14.46	¾	23.53	¾	32.60
5	5.67	13	14.74	21	23.81	29	32.89
5¼	5.95	¼	15.02	¼	24.09	¼	33.17
5½	6.23	½	15.31	½	24.38	½	33.46
5¾	6.52	¾	15.59	¾	24.66	¾	33.74
6	6.80	14	15.88	22	24.95	....	....
6¼	7.08	¼	16.16	¼	25.23	....	....
6½	7.37	½	16.45	½	25.51	....	....
6¾	7.65	¾	16.73	¾	25.80	....	....
7	7.94	15	17.01	23	26.08	....	....
7¼	8.22	¼	17.29	¼	26.36	....	....
7½	8.50	½	17.57	½	26.65	....	....
7¾	8.79	¾	17.86	¾	26.93	....	....
8	9.07	16	18.14	24	27.22	....	....

Table 300. Pressure Equivalents

Lb. per sq. in.	Lb. per sq. ft.	Lb. per circular in.	Kilogramme per sq. centimeter	1000 dynes per sq. centimeter	In. of mercury	Millimeters of mercury, 32° F.	Water pressure, feet, head	Water pressure, meters, head	Atmosphere
1.0	144.0	0.78540	0.0703	69.0	2.0376	51.63	2.307	0.703	0.06793
0.00694	1.0	0.00545	0.000488	0.00479	0.01415	0.3585	0.01602	0.00488	0.000472
1.273	183.3	1.0	0.08952	87.845	2.594	0.6573	2.937	0.8954	0.08648
14.223	2048.0	11.17	1.0	981.3	28.98	734.2	32.81	10.0	0.966
0.01449	2.086	0.01138	0.00101	1.0	0.02952	7.48	0.03343	0.01013	0.00984
0.4912	70.731	0.38579	0.03453	33.880	1.0	25.35	1.1334	0.3454	0.03336
0.01937	2.789	0.01521	0.001362	1.336	0.03947	1.0	0.04468	0.01362	0.001316
0.4335	62.425	0.34128	0.03048	29.91	0.882	22.38	1.0	0.30491	0.029448
1.422	204.76	1.1169	0.1	98.13	2.8975	73.42	3.281	1.0	0.0966
14.72	2119.68	11.562	1.035	1015.8	29.92	760.0	33.96	10.352	1.0

Table 301. Conversion Factors for Units of Pressure[3]

Unit	Ft. of water	Log	In. mercury	Log	Lbs. per sq. in.	Log	Lbs. per sq. ft.	Log
Lb. per sq. in.	2.308	0.3632	2.037	0.3090	1.0	0.0	144.0	2.1584
Lb. per sq. ft.	0.01603	8.2048–10	0.01414	8.1506–10	0.00694	7.8416–10	1.0	0.0
In. of mercury	1.133	0.0542	1.0	0.0	0.4910	9.6910–10	70.699	1.8494
Ft. of fresh water	1.0	0.0	0.8826	9.9458–10	0.4333	9.6368–10	62.4	1.7952
Ft. of sea water	1.025	0.0107	0.9047	9.9565–10	0.4442	9.6475–10	64.0	1.8062
Atmosphere	33.923	1.5305	29.942	1.4763	14.70	1.1673	2116.8	3.3257

Specific gravities used above; distilled water, 1.000; sea water, 1.025; mercury, 13.5956.

Table 302. Equivalent Measures and Weights of Water[4] at 4°C. (39.2°F.)

No.	U. S. gals.	Imperial gals.	Liters	Cubic meters
1	1.0	0.83321	3.7853	0.0037853
2	1.20017	1.0	4.54303	0.004543
3	0.264179	0.22012	1.0	0.001
4	264.179	220.117	1,000.0	1.0
5	0.119888	0.099892	0.453813	0.0004538
6	7.48055	6.23287	28.3161	0.0283161
7	0.004329	0.003607	0.0163866	0.0000164
8	0.0408	0.034	0.1544306	0.0001544
9	325,851	271,499	1,233,444	1,233.4438

No.	Lbs.	Cu. ft.	Cu. in.	Circular in. 1 ft. long	Acre-ft.
1	8.34112	0.13368	231.0	24.5096	0.00000307
2	10.0108	0.160439	277.274	29.4116	0.0000037
3	2.20355	0.035316	61.0254	6.4754	0.0000008
4	2,203.55	35.31563	61,025.4	6,475.44	0.000810
5	1.0	0.0160266	27.694	2.9411	0.00000037
6	62.3961	1.0	1,728.0	183.346	0.0000229
7	0.0361089	0.0005787	1.0	0.10613	0.00000001
8	0.340008	0.005454	9.4224	1.0	0.00000012
9	2,717,962	43,560	75,271,581	7,986,478	1.0

1 cu. ft. of ice weighs 57.2 to 57.5 lb. 1 cu. ft. of salt water weighs 63.9 to 64. 1 lb.

Table 303.* **Horsepower of Water (per Cu. Ft. per Sec.)**
Weight of Water Taken at 62.5 Lbs. per Cu. Ft.

Head, ft.	Horsepower		Head, ft.	Horsepower		Head, ft.	Horsepower	
	85 per cent. efficiency	100 per cent. efficiency		85 per cent. efficiency	100 per cent. efficiency		85 per cent. efficiency	100 per cent. efficiency
1	0.096591	0.113636	190	18.352273	21.590909	370	35.738637	42.045455
20	1.931818	2.272727	200	19.318182	22.727272	380	36.704546	43.181818
30	2.897727	3.409091	210	20.284091	23.863636	390	37.670455	44.318182
40	3.863636	4.545455	220	21.25	25.000000	410†	39.602273	46.590909
50	4.829545	5.681818	230	22.215909	26.136364	420	40.568182	47.727273
60	5.795455	6.818182	240	23.181818	27.272728	430	41.534091	48.863636
70	6.761364	7.954545	250	24.147727	28.409091	440	42.500000	50.000000
80	7.727273	9.090909	260	25.113637	29.545455	450	43.465909	51.136364
90	8.693182	10.227273	270	26.079546	30.681818	460	44.431818	52.272727
100	9.659091	11.363636	280	27.045455	31.818182	470	45.397728	53.409091
110	10.625	12.500000	290	28.011364	32.954545	480	46.363637	54.545455
120	11.590909	13.636363	300	28.977273	34.090909	490	47.329546	55.681818
130	12.556818	14.772727	310	29.943182	35.227273	520†	50.227273	59.090909
140	13.522727	15.909091	320	30.909091	36.363636	540	52.159091	61.363637
150	14.488636	17.045454	330	31.875000	37.500000	560	54.090909	63.636364
160	15.454546	18.181818	340	32.840909	38.636364	580	56.022727	65.909091
170	16.420455	19.318182	350	33.806818	39.772727	650	62.784091	73.863636
180	17.386364	20.454545	360	34.772727	40.909091	750†	72.443182	85.227273

* Horsepower = (discharge per sec. in lb. × head, ft. ÷ 550) × efficiency factor.
Example: A stream flow of 1000 cfs., and 100-ft. drop provides 9.659 × 1000 = 9659 hp. at 85 per cent. efficiency.
1 cu. ft. per min. falling 1 ft. = 0.00189 hp.
By moving decimal point in second and third columns of each group, horsepower for any head likely to be desired can be obtained nearly enough; closer results for some heads (*e.g.*, three or more significant figures) may be obtained by adding.
† Values for 400, 500, 600, 700, 800, etc., can be gotten from table by multiplying values for 40, 50, 60, 70, 80, respectively, by 10.

Water horsepower, or useful work done in pumping, equals capacity in gal. per min. multiplied by total head in ft., and divided by 3960: divisor is often taken at 4000 to simplify calculations (see p. 471). For sea water, add 2.6 per cent. to results.

Table 304. Equivalent Quantities of Water
Delivered in 24 Hrs., in 1 Hr., in 1 Min., and in 1 Sec.

Gals., 24 hrs.	Gals., 1 hr.	Gals., 1 min.	Cu. ft., per sec.	Gals., 24 hrs.	Gals., 1 hr.	Gals., 1 min.	Cu. ft. per sec.
2,500,000	104,166.6	1,736.0	3.8681	144,000	6,000.0	100.0	0.2228
2,000,000	83,333.3	1,388.0	3.0945	129,600	5,400.0	90.0	0.2005
1,500,000	62,500.0	1,041.7	2.3205	115,200	4,800.0	80.0	0.1782
1,000,000	41,666.6	694.4	1.5472	100,800	4,200.0	70.0	0.1559
950,000	39,583.3	659.7	1.4697	100,000	4,166.6	69.4	0.1547
900,000	37,500.0	625.0	1.3925	86,400	3,600.0	60.0	0.1337
850,000	35,416.6	590.2	1.3152	75,000	3,125.0	52.9	0.1160
800,000	33,333.3	555.5	1.2378	72,000	3,000.0	50.0	0.1114
750,000	31,250.0	520.8	1.1604	60,000	2,500.0	41.6	0.0928
700,000	29,166.6	486.1	1.0830	57,600	2,400.0	40.0	0.0811
650,000	27,083.3	451.3	1.0057	50,000	2,083.0	34.7	0.0774
600,000	25,000.0	416.7	0.9283	43,200	1,800.0	30.0	0.0668
550,000	22,916.6	381.9	0.8510	28,800	1,200.0	20.0	0.0446
500,000	20,833.3	347.2	0.7736	25,000	1,041.6	17.3	0.0387
450,000	18,750.0	312.5	0.6961	20,000	833.3	13.8	[illegible]309
400,000	16,666.6	277.7	0.6189	15,000	625.0	10.4	[illegible]232
350,000	14,583.3	243.0	0.5415	14,400	600.0	10.0	[illegible]223
300,000	12,500.0	208.3	0.4642	10,000	416.6	6.9	[illegible]55
250,000	10,416.7	173.6	0.3868	7,200	300.0	5.0	[illegible]11
200,000	8,333.3	138.8	0.3094	5,000	208.3	3.4	[illegible]77
150,000	6,250.0	104.1	0.2321	1,440	60.0	1.0	[illegible]22

Table 305. Cylindrical Tanks*—Capacity for Depth or Length of 1 Ft. and 1 In., Cu. Ft., Gals., and Lbs.

Depth	1 Ft.			1 In.			Depth	1 Ft.			1 In.			Depth	1 Ft.			1 In.		
Diam.	Cu. ft. †	Gals.	Lbs.	Cu. ft.	Gals.	Lbs.	Diam.	Cu. ft. †	Gals.	Lbs.	Cu. ft.	Gals.	Lbs.	Diam.	Cu. ft. †	Gals.	Lbs.	Cu. ft.	Gals.	Lbs.
2′-0″	3.1	24	194	0.262	1.97	16	5′-0″	19.6	147	1,227	1.6	12	102	8′-0″	50.3	376	3,141	4.2	31	262
1″	3.4	26	212	0.284	2.13	18	1″	20.3	152	1,268	1.7	13	106	1″	51.3	384	3,208	4.3	32	267
2″	3.7	28	231	0.307	2.31	19	2″	21.0	157	1,311	1.7	13	109	2″	52.4	392	3,274	4.4	33	273
3″	4.0	30	250	0.331	2.49	21	3″	21.7	162	1,353	1.8	13	113	3″	53.5	400	3,341	4.5	33	278
4″	4.3	32	268	0.356	2.68	22	4″	22.3	167	1,396	1.9	14	116	4″	54.5	408	3,409	4.5	34	284
5″	4.6	35	287	0.382	2.87	24	5″	23.0	172	1,440	1.9	14	120	5″	55.6	416	3,477	4.6	35	290
6″	4.9	37	306	0.409	3.04	25	6″	23.8	178	1,485	2.0	15	124	6″	56.8	424	3,546	4.7	35	295
7″	5.2	39	325	0.437	3.28	27	7″	24.5	183	1,530	2.0	15	127	7″	57.9	433	3,616	4.8	36	301
8″	5.6	42	349	0.465	3.50	29	8″	25.2	189	1,575	2.1	16	131	8″	59.0	441	3,687	4.9	37	307
9″	5.9	45	371	0.495	3.72	31	9″	26.0	194	1,623	2.2	16	135	9″	60.1	450	3,758	5.0	37	313
10″	6.3	47	394	0.525	3.95	33	10″	26.7	200	1,671	2.2	17	139	10″	61.3	459	3,830	5.1	38	319
11″	6.7	50	418	0.557	4.18	35	11″	27.5	206	1,718	2.3	17	143	11″	62.4	467	3,903	5.2	39	325
3′-0″	7.1	53	442	0.589	4.43	37	6′-0″	28.3	212	1,767	2.4	18	147	9′-0″	63.6	476	3,976	5.3	40	331
1″	7.5	56	467	0.622	4.78	39	1″	29.1	218	1,817	2.4	18	151	1″	64.8	485	4,050	5.4	40	337
2″	7.9	59	494	0.656	4.93	41	2″	29.9	224	1,867	2.5	19	156	2″	66.0	494	4,124	5.5	41	344
3″	8.3	62	519	0.681	5.20	43	3″	30.7	229	1,918	2.6	19	160	3″	67.2	503	4,200	5.6	42	350
4″	8.7	65	546	0.727	5.45	45	4″	31.5	236	1,969	2.6	20	164	4″	68.4	512	4,276	5.7	43	356
5″	9.2	69	573	0.757	5.74	48	5″	32.3	242	2,021	2.7	20	168	5″	69.6	521	4,353	5.8	43	363
6″	9.6	72	601	0.802	6.02	50	6″	33.2	248	2,074	2.8	21	173	6″	70.9	530	4,430	5.9	44	369
7″	10.1	75	630	0.84	6.31	52	7″	34.0	254	2,128	2.8	21	177	7″	72.1	539	4,508	6.0	45	376
8″	10.6	79	660	0.88	6.61	55	8″	34.9	261	2,182	2.9	22	182	8″	73.4	549	4,587	6.1	46	382
9″	11.0	82	690	0.92	6.9	57	9″	35.8	268	2,236	3.0	22	186	9″	74.7	558	4,666	6.2	46	389
10″	11.5	86	721	0.968	7.22	60	10″	36.7	275	2,292	3.1	23	191	10″	75.9	568	4,746	6.3	47	395
11″	12.0	90	753	1.00	7.52	63	11″	37.6	281	2,348	3.1	23	196	11″	77.2	578	4,827	6.4	48	402
4′-0″	12.6	94	786	1.0	8	65	7′-0″	38.5	288	2,405	3.2	24	200	10′-0″	79	590	4,910	7	49	410
1″	13.1	98	819	1.1	8	68	1″	39.4	295	2,463	3.3	25	205	3″	82	620	5,160	7	50	430
2″	13.6	102	853	1.1	8	71	2″	40.3	302	2,521	3.4	25	210	6″	87	650	5,410	7	50	450
3″	14.2	106	887	1.2	9	74	3″	41.3	309	2,580	3.4	26	215	9″	91	680	5,670	8	60	470
4″	14.7	110	922	1.2	9	77	4″	42.2	316	2,640	3.5	26	220	11′-0″	95	710	5,940	8	60	490
5″	15.3	115	958	1.3	10	80	5″	43.2	323	2,700	3.6	27	225	3″	99	740	6,210	8	60	520
6″	15.9	119	994	1.3	10	83	6″	44.2	330	2,761	3.7	27	230	6″	104	780	6,490	9	70	540
7″	16.5	123	1,031	1.4	10	86	7″	45.2	338	2,823	3.8	28	235	9″	108	810	6,770	9	70	560
8″	17.1	128	1,069	1.4	11	89	8″	46.2	346	2,885	3.8	29	240	12′-0″	113	850	7,070	9	70	590
9″	17.7	133	1,108	1.5	11	92	9″	47.2	353	2,948	3.9	29	246	3″	118	880	7,370	10	70	610
10″	18.3	137	1,147	1.5	11	96	10″	48.2	360	3,012	4.0	30	251	6″	123	920	7,670	10	80	640
11″	19.0	142	1,187	1.6	12	99	11″	49.2	368	3,076	4.1	31	256	9″	128	950	7,980	11	80	670

* Applies to pipes also. † Numbers in these columns are also areas in sq. ft. corresponding to diam.

Table 305. Cylindrical Tanks—Capacity for Depth or Length of 1 Ft. and 1 In., Cu. Ft., Gals., and Lbs.—(*Continued*)

Depth Diam.	1 Ft. Cu. ft.*	1 Ft. Gals.	1 Ft. Lbs.	1 In. Cu. ft.	1 In. Gals.	1 In. Lbs.
13′–0″	133	990	8.290	11	80	690
3″	138	1,030	8,620	11	90	720
6″	143	1,070	8,940	12	90	750
9″	148	1,110	9,280	12	90	770
14′–0″	154	1,150	9,620	13	100	800
3″	159	1,190	9,970	13	100	830
6″	165	1,240	10,320	14	100	860
9″	171	1,280	10,680	14	110	890
15′–0″	177	1,320	11,040	15	110	920
3″	183	1,370	11,420	15	110	950
6″	189	1,410	11,790	16	120	980
9″	195	1,460	12,180	16	120	1,010
16′–0″	201	1,500	12,570	17	120	1,050
3″	207	1,550	12,960	17	130	1,080
6″	214	1,600	13,360	18	130	1,110
9″	220	1,650	13,770	18	140	1,150
17′–0″	227	1,700	14,190	19	140	1,180
3″	234	1,750	14,610	20	150	1,220
6″	240	1,800	15,030	20	150	1,250
9″	247	1,850	15,470	21	150	1,290
18′–0″	254	1,900	15,910	21	160	1,330
3″	262	1,960	16,350	22	160	1,360
6″	269	2,010	16,800	22	170	1,400
9″	276	2,060	17,260	23	170	1,440
19′–0″	284	2,120	17,720	24	180	1,480
3″	291	2,180	18,190	24	180	1,520
6″	299	2,230	18,670	25	190	1,560
9″	306	2,290	19,150	25	190	1,600
20′–0″	315	2,350	19,700	25	200	1,600
6″	330	2,450	20,600	30	200	1,700
[illegible]	345	2,600	21,600	30	220	1,800

Depth Diam.	1 Ft. Cu. ft.*	1 Ft. Gals.	1 Ft. Lbs.	1 In. Cu. ft.	1 In. Gals.	1 In. Lbs.
21′–6″	365	2,700	22,700	30	220	1,900
22′–0″	380	2,850	23,800	30	240	2,000
6″	400	3,000	24,900	35	250	2,100
23′–0″	415	3,100	25,900	35	260	2,200
6″	435	3,250	27,100	35	270	2,300
24′–0″	450	3,400	28,200	35	280	2,400
6″	470	3,500	29,400	40	290	2,500
25′–0″	490	3,650	30,700	40	300	2,600
6″	510	3,800	31,900	40	320	2,700
26′–0″	530	3,950	33,200	45	330	2,800
6″	550	4,150	34,500	45	350	2,900
27′–0″	575	4,300	35,700	50	360	3,000
6″	595	4,450	37,100	50	370	3,100
28′–0″	615	4,600	38,500	50	380	3,200
6″	640	4,750	39,900	55	390	3,300
29′–0″	660	4,950	41,300	55	410	3,400
6″	685	5,100	42,700	55	430	3,600
30′–0″	705	5,300	44,200	60	440	3,700
6″	730	5,450	45,700	60	450	3,800
31′–0″	755	5,650	47,200	65	470	3,900
6″	780	5,850	48,700	65	490	4,100
32′–0″	805	6,000	50,300	65	500	4,200
6″	830	6,200	51,900	70	520	4,300
33′–0″	855	6,400	53,400	70	530	4,400
6″	880	6,600	55,100	75	550	4,600
34′–0″	910	6,800	56,800	75	570	4,700
6″	935	7,000	58,400	80	580	4900
35′–0″	960	7,200	60,100	80	600	5000
6″	990	7,400	61,900	85	620	5200
36′–0″	1020	7,650	63,600	85	640	5300
6″	1045	7,800	65,400	85	650	5500

Depth Diam.	1 Ft. Cu. ft.*	1 Ft. Gals.	1 Ft. Lbs.	1 In. Cu. ft.	1 In. Gals.	1 In. Lbs.
37′–0″	1075	8,050	67,200	90	670	5600
6″	1105	8,250	69,000	90	690	5800
38′–0″	1135	8,500	70,900	95	710	5900
6″	1165	8,700	72,800	95	730	6100
39′–0″	1195	8,950	74,700	100	750	6200
6″	1225	9,150	76,600	100	760	6400
40′–0″	1255	9,400	78,600	105	780	6500
6″	1290	9,650	80,500	105	800	6700
41′–0″	1320	9,850	82,500	110	820	6900
6″	1355	10,150	84,600	115	850	7100
42′–0″	1385	10,350	86,600	115	860	7200
6″	1420	10,600	88,700	120	880	7400
43′–0″	1450	10,850	90,800	120	900	7600
6″	1485	11,100	92,900	125	930	7700
44′–0″	1520	11,400	95,100	125	950	7900
6″	1555	11,650	97,200	130	970	8,100
45′–0″	1590	11,900	99,400	135	990	8,300
6″	1625	12,150	101,600	135	1010	8,500
46′–0″	1660	12,450	103,900	140	1040	8,700
6″	1700	12,700	106,100	140	1060	8,800
47′–0″	1735	13,000	108,400	145	1080	9,000
6″	1770	13,250	110,800	150	1100	9,200
48′–0″	1810	13,550	113,100	150	1130	9,400
6″	1845	13,800	115,400	155	1150	9,600
49′–0″	1885	14,100	117,900	155	1170	9,800
6″	1925	14,400	120,200	160	1200	10,000
50′–0″	1965	14,700	122,700	165	1230	10,200

1 cu. ft. = 7.48 gals. and 62.5 lbs. in above tables. Errors of about 2 per cent., due to rounding number.

* Numbers in these columns are also areas in sq. ft. corresponding to diam.

Table 306. Tanks—Capacities in Cubic Feet, Gallons, and Pounds, for Depths of 1 Ft. and 1 In., for Horizontal Areas in Square Feet*

Maximum Error Due to Round Numbers = 0.26 Per Cent.

Area, sq. ft.	1 Ft. depth			1 In. depth		
	Cu. ft.	Gals.	Weight, lbs.	Cu. ft.	Gals.	Weight, lbs.
0.1	0.1	0.748	6.25	0.0083	0.06	0.52
0.2	0.2	1.5	12.5	0.017	0.12	1.04
0.3	0.3	2.2	18.7	0.025	0.19	1.56
0.4	0.4	3.0	25.0	0.033	0.25	2.08
0.5	0.5	3.7	31.2	0.042	0.31	2.60
0.6	0.6	4.5	37.5	0.050	0.37	3.12
0.7	0.7	5.2	43.7	0.058	0.44	3.65
0.8	0.8	6.0	50.0	0.067	0.50	4.17
0.9	0.9	6.7	56.2	0.075	0.56	4.69
1.0	1.0	7.5	62.5	0.083	0.62	5.21
1.1	1.1	8.2	69	0.092	0.68	5.7
1.2	1.2	9.0	75	0.100	0.75	6.2
1.3	1.3	9.7	81	0.108	0.81	6.8
1.4	1.4	10.5	87	0.117	0.87	7.3
1.5	1.5	11.2	94	0.125	0.93	7.8
1.6	1.6	12.0	100	0.133	1.00	8.3
1.7	1.7	12.7	106	0.142	1.06	8.9
1.8	1.8	13.5	112	0.150	1.12	9.4
1.9	1.9	14.2	119	0.158	1.18	9.9
2.0	2.0	15.0	125	0.167	1.25	10.4
2.1	2.1	15.7	131	0.175	1.31	10.9
2.2	2.2	16.4	137	0.183	1.37	11.5
2.3	2.3	17.2	144	0.192	1.44	12.0
2.4	2.4	17.9	150	0.200	1.50	12.5
2.5	2.5	18.7	156	0.208	1.56	13.0
2.6	2.6	19.4	162	0.217	1.62	13.5
2.7	2.7	20.2	169	0.225	1.69	14.0
2.8	2.8	20.9	175	0.233	1.75	14.6
2.9	2.9	21.7	181	0.242	1.81	15.1
3.0	3.0	22.4	187	0.250	1.87	15.6
3.1	3.1	23.2	194	0.258	1.93	16.1
3.2	3.2	23.9	200	0.267	1.99	16.6
3.3	3.3	24.6	206	0.275	2.06	17.2
3.4	3.4	25.4	212	0.283	2.12	17.7
3.5	3.5	26.2	219	0.292	2.18	18.2
3.6	3.6	26.9	225	0.300	2.24	18.7
3.7	3.7	27.6	231	0.308	2.31	19.3
3.8	3.8	28.4	237	0.317	2.37	19.8
3.9	3.9	29.2	244	0.325	2.43	20.3
4.0	4.0	29.9	250	0.333	2.49	20.8
4.1	4.1	30.7	256	0.342	2.55	21.3
4.2	4.2	31.4	262	0.350	2.61	21.9
4.3	4.3	32.1	269	0.358	2.68	22.4
4.4	4.4	32.9	275	0.367	2.74	22.9
4.5	4.5	33.6	281	0.375	2.80	23.4
4.6	4.6	34.4	287	0.383	2.86	23.9
4.7	4.7	35.1	294	0.392	2.93	24.5
4.8	4.8	35.9	300	0.400	2.99	25.0
4.9	4.9	36.6	306	0.408	3.05	25.5
5.0	5.0	37.4	312	0.417	3.12	26.0
5.1	5.1	38.1	319	0.425	3.18	26.5
5.2	5.2	38.9	325	0.433	3.24	27.0
5.3	5.3	39.6	331	0.442	3.31	27.6
5.4	5.4	40.4	337	0.450	3.37	28.1
5.5	5.5	41.1	344	0.458	3.43	28.6
5.6	5.6	41.9	350	0.467	3.49	29.1
5.7	5.7	42.6	356	0.475	3.56	29.7
5.8	5.8	43.4	362	0.483	3.62	30.2
5.9	5.9	44.1	369	0.492	3.68	30.7
6.0	6.0	44.9	375	0.500	3.74	31.2
[illegible]	6.1	45.6	381	0.508	3.80	31.7
[illegible]	6.2	46.4	387	0.517	3.86	32.3
[illegible]	6.3	47.1	394	0.525	3.93	32.8
[illegible]	6.4	47.9	400	0.533	3.99	33.3

*[illegible]l also for pipes per ft. or in. of length.

Table 306. Tanks—Capacity for Depths of 1 Ft. and 1 In.—(*Continued*)

Area, sq. ft.	1 Ft. depth			1 In. depth		
	Cu. ft.	Gals.	Weight, lbs.	Cu. ft.	Gals.	Weight, lbs.
6.5	6.5	48.6	406	0.542	4.05	33.8
6.6	6.6	49.4	412	0.550	4.11	34.3
6.7	6.7	50.1	419	0.558	4.18	34.9
6.8	6.8	50.9	425	0.567	4.24	35.4
6.9	6.9	51.6	431	0.575	4.30	35.9
7.0	7.0	52.4	437	0.583	4.36	36.5
7.1	7.1	53.1	444	0.592	4.42	37.0
7.2	7.2	53.8	450	0.600	4.48	37.5
7.3	7.3	54.6	456	0.608	4.55	38.0
7.4	7.4	55.3	462	0.617	4.61	38.5
7.5	7.5	56.1	469	0.625	4.67	39.1
7.6	7.6	56.8	475	0.633	4.73	39.6
7.7	7.7	57.6	481	0.642	4.80	40.1
7.8	7.8	58.3	487	0.650	4.86	40.6
7.9	7.9	59.1	494	0.658	4.92	41.1
8.0	8.0	59.8	500	0.667	4.99	41.7
8.1	8.1	60.6	506	0.675	5.05	42.2
8.2	8.2	61.3	512	0.683	5.11	42.7
8.3	8.3	62.1	519	0.692	5.18	43.2
8.4	8.4	62.8	525	0.700	5.24	43.7
8.5	8.5	63.6	531	0.708	5.30	44.3
8.6	8.6	64.3	537	0.717	5.36	44.8
8.7	8.7	65.1	544	0.725	5.43	45.3
8.8	8.8	65.8	550	0.733	5.49	45.8
8.9	8.9	66.6	556	0.742	5.55	46.3
9.0	9.0	67.3	562	0.750	5.61	46.9
9.1	9.1	68.1	569	0.758	5.67	47.4
9.2	9.2	68.8	575	0.767	5.73	47.9
9.3	9.3	69.6	581	0.775	5.80	48.4
9.4	9.4	70.3	587	0.782	5.86	49.0
9.5	9.5	71.1	594	0.792	5.92	49.5
9.6	9.6	71.8	600	0.800	5.98	50.0
9.7	9.7	72.6	606	0.808	6.05	50.5
9.8	9.8	73.3	612	0.817	6.11	51.0
9.9	9.9	74.1	619	0.825	6.17	51.6
10.0	10.0	74.8	625	0.833	6.23	52.0
20.0	20.0	150	1,250	1.67	12.5	100
30.0	30.0	224	1,870	2.50	18.7	160
40.0	40.0	299	2,500	3.33	24.7	210
50.0	50.0	374	3,120	4.17	31.2	260
60.0	60.0	449	3,750	5.00	37.4	310
70.0	70.0	524	4,370	5.83	43.6	360
80.0	80.0	598	5,000	6.67	49.8	420
90.0	90.0	673	5,620	7.50	56.1	470
100.0	100.0	748	6,250	8.33	62.3	520
110.0	110.0	823	6,870	9.17	68.5	570
120.0	120.0	898	7,500	10.00	74.8	620
130.0	130.0	972	8,120	10.83	81.0	680
140.0	140.0	1,047	8,750	11.67	87.0	730
150.0	150.0	1,122	9,380	12.50	93.4	780
160.0	160.0	1,197	10,000	13.33	99.7	830
170.0	170.0	1,272	10,600	14.17	105.9	890
180.0	180.0	1,346	11,250	15.00	112.1	940
190.0	190.0	1,421	11,850	15.83	118.4	990
200.0	200.0	1,496	12,500	16.67	124.6	1,040
210.0	210.0	1,570	13,100	17.5	131	1,090
220.0	220.0	1,650	13,750	18.3	137	1,150
230.0	230.0	1,720	14,350	19.2	144	1,200
240.0	240.0	1,800	15,000	20.0	150	1,250
250.0	250.0	1,870	15,600	20.8	156	1,300
260.0	260.0	1,950	16,250	21.7	162	1,350
270.0	270.0	2,020	16,850	22.5	169	1,410
280.0	280.0	2,090	17,500	23.3	175	1,460
290.0	290.0	2,170	18,100	24.2	181	1,510
300.0	300.0	2,240	18,750	25.0	187	1,560
310.0	310.0	2,350	19,400	25.8	193	1,6[illegible]
320.0	320.0	2,390	20,000	26.7	199	1,6[illegible]
330.0	330.0	2,470	20,600	27.5	206	1,7[illegible]
340.0	340.0	2,540	21,250	28.3	212	1,7[illegible]

Table 306. Tanks—Capacity for Depths of 1 Ft. and 1 In.—(*Concluded*)

Area, sq. ft.	1 Ft. depth			1 In. depth		
	Cu. ft.	Gals.	Weight, lbs.	Cu. ft.	Gals.	Weight, lbs.
350.0	350.0	2,620	21,850	29.2	218	1,820
360.0	360.0	2,690	22,500	30.0	224	1,870
370.0	370.0	2,770	23,100	30.8	231	1,930
380.0	380.0	2,840	23,750	31.7	237	1,980
390.0	390.0	2,920	24,350	32.5	243	2,030
400.0	400.0	2,990	25,000	33.3	249	2,080
410.0	410.0	3,070	25,600	34.2	255	2,130
420.0	420.0	3,140	26,250	35.0	261	2,190
430.0	430.0	3,220	26,850	35.8	268	2,240
440.0	440.0	3,290	27,500	36.7	274	2,290
450.0	450.0	3,360	28,100	37.5	280	2,340
460.0	460.0	3,440	28,750	38.3	286	2,390
470.0	470.0	3,510	29,400	39.2	293	2,450
480.0	480.0	3,590	30,000	40.0	299	2,500
490.0	490.0	3,660	30,600	40.8	305	2,550
500.0	500.0	3,750	31,250	41.7	310	2,600
510.0	510.0	3,800	31,850	42.5	320	2,650
520.0	520.0	3,900	32,500	43.3	325	2,700
530.0	530.0	3,950	31,100	44.2	330	2,760
540.0	540.0	4,050	33,750	45.0	335	2,810
550.0	550.0	4,100	34,350	45.8	345	2,860
560.0	560.0	4,200	35,000	46.7	350	2,910
570.0	570.0	4,250	35,600	47.5	355	2,960
580.0	580.0	4,350	36,250	48.3	360	3,020
590.0	590.0	4,400	36,850	49.2	370	3,070
600.0	600.0	4,500	37,500	50.0	375	3,120
610.0	610.0	4,550	38,100	50.8	380	3,170
620.0	620.0	4,650	38,750	51.7	385	3,230
630.0	630.0	4,700	39,400	52.5	395	3,280
640.0	640.0	4,800	40,000	53.3	400	3,330
650.0	650.0	4,900	40,600	54.2	405	3,380
660.0	660.0	4,950	41,250	55.0	410	3,430
670.0	670.0	5,000	41,850	55.8	420	3,490
680.0	680.0	5,100	42,500	56.7	425	3,540
690.0	690.0	5,150	43,100	57.5	430	3,590
700.0	700.0	5,250	43,750	58.3	435	3,650
710.0	710.0	5,300	44,350	59.2	445	3,700
720.0	720.0	5,400	45,000	60.0	450	3,750
730.0	730.0	5,450	45,600	60.8	455	3,800
740.0	740.0	5,550	46,250	61.7	460	3,850
750.0	750.0	5,600	46,850	62.5	470	3,910
760.0	760.0	5,700	47,500	63.3	475	3,960
770.0	770.0	5,750	48,100	64.2	480	4,010
780.0	780.0	5,850	48,750	65.0	485	4,060
790.0	790.0	5,900	49,400	65.8	490	4,110
800.0	800.0	6,000	50,000	66.7	500	4,170
810.0	810.0	6,050	50,600	67.5	505	4,220
820.0	820.0	6,150	51,250	68.3	510	4,270
830.0	830.0	6,200	51,850	69.2	515	4,320
840.0	840.0	6,300	52,500	70.0	525	4,370
850.0	850.0	6,350	53,100	70.8	530	4,430
860.0	860.0	6,450	53,750	71.7	535	4,480
870.0	870.0	6,500	54,350	72.5	540	4,530
880.0	880.0	6,600	55,000	73.3	550	4,580
890.0	890.0	6,650	55,650	74.2	555	4,630
900.0	900.0	6,750	56,250	75.0	560	4,690
910.0	910.0	6,800	56,850	75.8	565	4,740
920.0	920.0	6,900	57,500	76.7	575	4,790
930.0	930.0	6,950	58,100	77.5	580	4,840
940.0	940.0	7,000	58,750	78.3	585	4,900
950.0	950.0	7,100	59,400	79.2	590	4,950
960.0	960.0	7,200	60,000	80.0	600	5,000
970.0	970.0	7,250	60,650	80.8	605	5,050

Above table is based on 1 cu. ft. of water weighing 62.5 lb. and containing 7.48 gal.

Example of use: Cistern 18 ft. × 50 ft. × 10 ft. 1 in. deep contains, how many gal.? 18 × 50 = 900. Entering table under "Area" at 900, find 6750 gal. per ft. depth; 560 per in. Capacity = 67,500 [illegible] 560 = 68,060 gal. The above weights are exclusive of weight of tank. For circular tanks obtai[illegible]as from tables of circles or compute.

Bibliography, Chapter 37. Capacity and Conversion Tables

1. [illegible]io *Eng. Soc.*, Jan., 1912, p. 78. 2. *Water Supply and Irrigation Paper* 425-C, 1917, p. 72. 3. [illegible]es and Safford: Hydraulics, 1911, p. 6. 4. "Notes on Hydrology," D. W. Mead, 1904, p. 12.

CHAPTER 38

MISCELLANY

COLD PERIODS IN UNITED STATES[1]

1717. "Great Snow," Feb. 19–24; 6 ft. deep, Boston.

1741. Intense cold. Long Island Sound frozen. Connecticut River ice would carry horses, Apr. 1; 16 ft. of ice in Hudson River.*

1765–6. Hudson frozen over at Paulus Hook; 26°F., St. Johns River, Fla.

1780 Coldest, except 1856. Chesapeake Bay frozen over from head to Potomac; −6° at Williamsburg, Va. Ice solid enough to support pedestrians. Ice entirely between New York and Staten Island, and on Long Island Sound. Troops crossed from New Jersey to Staten Island on ice. Bayou St. John, New Orleans, was frozen. Delaware River frozen over from Dec. 1 to Mar. 14; ice 2 or 3 ft. thick.

1783. −12° at Philadelphia; Hartford, Conn., −20°.

1787–8. Ground frozen at Savannah, 20°F.

1792. Ohio River at Wheeling frozen over for 40 days.

1796–7. Ohio and Mississippi rivers and confluents frozen to their junction; 17° Charleston; −19° Cincinnati.

1800. Snow 18 in. deep in Georgia. Snowfall 5 in. deep at St. Marys River, Fla. Natchez, Miss., 6°. Sleet and heavy frosts in Louisiana.

1816. At Williamstown, Mass., frost in each summer month. Snow, June 8 in Vermont. Frost in Philadelphia, July.

1820–1. Hudson at Paulus Hook covered with ice (fourth time in 100 years).

1825–6. −27°, Portland, Maine; −24°, Amherst; −18°, Springfield, Mass.; 0°, Washington; 14° in Louisiana; Montreal, −38°.

1831. Mississippi River frozen for 130 mi. below mouth of Ohio.

1835. Long Island Sound closed; Boston harbor, nearly so.

1835. Jan. 4: Bangor, Maine, −40°; Bath, Maine, −40°; Portland, Maine, −21°; Montpelier, Vt., −40°; Concord, N. H., −35°; Boston, −15°; Pittsfield, Mass., −32°; Albany, −32°; Poughkeepsie, −35°; New York, −5°; Lancaster, Pa., −22°; Washington, −16°; Baltimore, −10°. Jan. 5: New Haven, Conn., −27°; Providence, −16°; Philadelphia, −6°. Feb. 8: Chicago, −22°; St. Louis, −25°; Cincinnati, −18°; Nashville, −10°; Huntsville, Ala., −9°; Natchez, Miss., 0°; Baton Rouge, 10°; Jacksonville, 8°; Richmond, −6°; Savannah, 3°; Augusta, −2°.

1851–2. St. Louis, −14°; New Orleans, 17°; Pensacola, 10°; Washington, −7°; New York, −8°. Potomac at Washington frozen over for 3 weeks. East River frozen; crossed from Jan. 20–24.

* von Riedesel is said to have conveyed cannon on ice to the Battery from Staten Island, New York.[2]

1854. Salt Lake City, −14°; Fort Dallas, Ore., −15°; Fort Defiance, N. M., −20°; San Francisco, 27°.

1856. Pittsburgh, −18°. Ice formed in Louisiana. Severest winter of 50 years in Mississippi Valley.

1864. Cincinnati, −5°; Fort Laramie, Wyo., −40°.

1866–67. Ferries stopped in East River, New York City.[2]

Table 307. Altitudes, Barometer and Wind*

State, country or territory	Place	Altitude, ft. above sea level	Barometer, inches of mercury			Velocity of wind, miles per hr.†	
			Annual mean	Aver. annual max.	Aver. annual min.	Annual mean (20 yrs.)	Max. recorded 1909–14
Alabama	Birmingham	700	29.32	29.77	28.63	10	48
Alaska	Sitka	90	29.79	29.99	29.48	---	---
Arkansas	Little Rock	357	29.66	29.80	29.55	6	59
Arizona	Phoenix	1108	28.75	28.89	28.62	---	40
Arizona	Yuma	141	29.73	29.92	29.58	---	44
California	Los Angeles	338	29.62	29.73	29.52	4	42
California	San Francisco	155	29.86	29.99	29.75	10	64
Colorado	Denver	5291	24.71	24.82	24.58	7	75
Connecticut	New Haven	106	29.91	30.04	29.79	---	58
Cuba	Havana	57	29.92	30.07	29.82	---	---
District of Columbia	Washington	112	29.94	30.01	29.82	5	68
Florida§	Jacksonville	43	30.01	30.13	29.91	8	75
Georgia	Atlanta	1174	28.83	28.93	28.74	10	66
Georgia	Savannah	65	29.99	30.11	29.89	7	88
Idaho	Boise City	2739	27.17	27.33	27.06	4	55
Illinois	Cairo	356	29.67	29.80	29.52	8	54
Illinois	Chicago	823	29.13	29.23	29.03	17	84
Illinois	Springfield	644	29.35	29.47	29.25	9	48
Indiana	Indianapolis	822	29.16	29.27	29.06	10	60
Iowa	Davenport	606	29.37	29.49	29.26	---	48
Iowa	Dubuque	698	29.27	29.38	29.16	6	60
Kansas	Dodge City	2509	27.38	27.48	27.27	11	75
Kansas	Topeka	983	---	---	---	---	58
Louisiana	New Orleans	51	29.98	30.10	29.87	8	66
Maine	Portland	103	29.87	30.00	29.75	5	61
Maryland	Baltimore	123	29.92	30.05	29.80	---	70
Massachusetts	Boston	125	29.88	30.00	29.75	11	72
Michigan	Detroit	730	29.23	29.33	29.13	11	86
Michigan	Sault St. Marie	614	29.31	29.41	29.24	8	52
Minnesota	Duluth	1133†	29.23	29.33	29.13	14	78
Minnesota	St. Paul	837	29.09	29.20	28.98	8	102
Missouri	St. Louis	567	29.43	29.56	29.33	11	80
Nebraska	North Platte	2821	27.06	27.15	26.96	10	---
Nebraska	Omaha	1105	28.84	28.97	28.73	9	66
New Mexico	Santa Fe	7013	23.25	23.37	23.12	7	53
New York	Albany	85	29.92	30.05	29.80	6	70
New York	Buffalo	767‡	29.18	29.28	29.08	11	92
New York	New York	314	29.69	29.82	29.57	9	96
Ohio	Cincinnati	628	29.38	29.49	29.28	7	59
Ohio	Cleveland	762	29.21	29.30	29.11	11	73
Oklahoma	Oklahoma City	1214	28.72	28.85	28.60	11	72
Oregon	Portland	154	29.90	30.03	29.78	6	39
Pennsylvania	Harrisburg	374	29.65	29.77	29.53	7	46
Pennsylvania	Philadelphia	117	29.93	30.06	29.81	10	75
Pennsylvania	Pittsburg	842	29.14	29.24	29.04	7	69

* Compiled from U. S. Weather Bureau records. ‡ Instrument previously at 702.
† Those above 75 mi. per hr. are classed as "hurricanes."
§ Florida hurricane, Sept. 20, 1926, wind velocity attained 152 miles per hour (*E. N. R.*, Oc[illegible], 1926, p. 639).

Table 307. Altitudes, Barometer and Wind.—(*Continued*)

State, country or territory	Place	* Altitude, ft. above sea level	Barometer, inches of mercury			Velocity of wind, miles per hr.	
			Annual mean	Aver. annual max	Aver. annual min.	Annual mean (20 yrs.)	Max.† recorded 1909–14
Porto Rico	San Juan	82	29.90	29.97	29 81	- - -	72
South Carolina	Charleston	48	30 02	30.14	29 91	10	94
South Dakota	Pierre	1572	28.30	28.42	28.20	9	65
Tennessee	Chattanooga	762	29.26	29.38	29.16	6	55
Tennessee	Knoxville	1004	29.02	29.13	28 92	6	42
Tennessee	Memphis	397	29.63	29.76	29.51	8	72
Texas	Galveston	54	29.97	30.11	29 86	10	84
Utah	Salt Lake City	4366	25 62	25.74	25 51	6	66
Vermont	Burlington	268	29 71	29.85	29.55	- - -	66
Virginia	Richmond	144	29 91	30.04	29 78	8	61
Washington	Seattle	123	29 92	30 03	29 79	6	64
Washington	Spokane	1943	27.96	28.11	27.86	4	52

* Height of observing instruments

† Compiled by searching U S Weather Bureau records for these years, records give for each month the maximum velocity for that month

1867. December. −10°, Wytheville, Va.; St. Paul, −39°.

1872. December. Chicago, −23°, Lunenburg, Vt., −45°;

1875. January. Ft. Garland, Colo., −40°; Logansport, Ind., −30°; New Haven, −14°.

1875. Jan. 9. −6°, New York; −12°, Pittsburgh, −3°, Washington.

1904–5. 5 to 8° below normal for Eastern States.

1917. Dec. 30. −13°, New York: −5° Pittsburgh; −3°, Washington.

Lamé's General Formula for Thick, Hollow Cylinder.

$$s = \left[r_1^2P_1 - r_2^2P_2 + \frac{r_1^2r_2^2}{x^2}(P_1 - P_2) \right] \div (r_2^2 - r_1^2)$$

s = tangential unit stress at distance x from axis of cylinder.
r_1 = internal radius of cylinder.
r_2 = external radius of cylinder.
P_1 = internal unit stress.
P_2 = external unit stress.

Truck Loads on Valve Chamber Covers. Fifty-ton girders on a truck weighing 16 tons (one of the largest in New York City) broke through 24- by 24-in. cast-iron manhole covers on Broadway (March 1912), presumably due to impact. The major part of the load was on the rear axle and was computed to be 22 tons per wheel. A 68-ton girder for the Pennsylvania terminal was transported through city streets; also the 70-ton girders for the Woolworth building. Sometimes it is cheaper to ignore such abnormal loads, and replace broken covers. An 80-ton girder gives a wheel load of about 29 tons. The 24- by 24-in. covers of the Metropolitan Street Railway Co. mentioned above were capable of sustaining a static load of 30 tons. Sections of the German mine-laying submarine UC5. were trucked through New York City streets in connection with Liberty bond campaigns. Transported on special truck weighing 15 tons; largest sections weighed 50, 55, and 60 tons. W[illegible]

of special truck had 14-in. tread. Few manhole covers were broken, one load running over several on Manhattan Avenue and 125th Streets without causing any damage.[5]

Earthquakes—Effects on Waterworks Structures.* Committee of engineers, San Francisco, April, 1906.[6] Conclusions from extended observations of damage done: (1) Greater attention should be given to avoidance of threatening geologic faults, in locating important waterworks structures. (2) Skilfully designed and well-built earth dams were proved of great stability, deserving increased confidence. (3) Concrete dams of gravity section are able to withstand shocks of great severity without damage. (4) Distributing reservoirs, pumping machinery, elevated tanks, and standpipes, if secure as to foundations and intelligently designed according to best practice, will withstand shocks sufficient to destroy most buildings. (5) Pipes or conduits of any character are almost certain to fail when intersected by a plane of large movement, whether faulting, sliding, or settling. In planning distributing systems for cities, important supply pipes should be located along routes carefully selected for stability; areas liable to serious disturbances should be so segregated as to be quickly separable from remainder of system. (6) Distributing pipe systems should have abundance of suitably placed, well-maintained valves, conspicuously marked in field and clearly recorded, for isolating pipes and sections; critically important supply mains should be duplicated along widely separated routes; ample supplies of water should be maintained close to centers of use; structures liable to shock should be of substantial types. Pipe lines in firm ground sustained no considerable damage. Marshy, filled, and soft alluvial soils constitute danger spots. If they cannot be wholly avoided for important mains, partial or entire immunity may be found in pile foundations extending well below compressible soil, and heavy flexible joints across lines of probable separation.

The Japanese earthquake of 1923 is being studied by a committee of Am. Soc. C. E. Most of the joints in distribution system were loosened. Artesian wells were but slightly affected.[29]

Population studies† are required in forecasting water-supply requirements. The method commonly used in reports on future water demands is to plot the decennial census figures furnished by the U. S. Census Bureau, Department of the Interior, for the community under investigation, and also for other larger communities of a similar character. Curves are then constructed to indicate roughly the past rate of growth; the probability is that unless pestilence, sudden migratory movement, or change of area intervene, the community will continue its past rate of growth into the future. This method lacks the finesse of those used by actuaries, but suffices for water-supply studies. Plot years as abscissas, and populations as ordinates. By a parallel shifting of the curves of the larger towns over the curve of the town under investigation, so that the curves coincide for the last census of said town, the inve[illegible]ator gets an idea of the trend of similar towns as they increased in size, and [illegible] be guided accordingly in prognosticating future growth.

* [illegible] also, "Notes on the Guatemala Earthquakes and Earthquake-proof Construction," by Vil[illegible]joy, *T. A. S. C. E.*, Vol. 83, 1919, p. 1689–1712, and Hadley in Proc. Am. Conc. Inst., Vol. 20, [illegible]

† [illegible] also Johnson, *J. N. E. W. W. A.*, Vol. 28, 1914, p. 152.

A second method* used involves incremental increases and is based on the theory that increase in the increments are constant—in other words, that the curve of population is of the second degree. Find the increase for each decade and find the difference in the increase for each pair of decades. By averaging both the increase for each decade and the increment of increase for successive pairs of decades, figures are obtained which properly applied to the first figure of the series will produce the last figure; this can be extended into the future. In using this method, the population at the end of the first future decade is obtained by adding to the present figure the average increase plus the average increment; at the end of the second future decade, the population will be that at the end of the first future decade plus the average increase plus twice the increment.[7]

WATERWORKS FINANCING

Ownership. Public vs. Private. In reporting to San Francisco Chamber of Commerce advocating purchase of property of Spring Valley Water Co., J. Waldo Smith[9] said:

> Water supply is the one public utility concerning which no questions are raised as to the wisdom of municipal ownership and operation A large proportion of all water supplies in the country, both large and small, are municipally owned. In general, the municipality has rendered good service in the operation and maintenance of this utility.

In developing a water supply for a young community the initiative of the private investor is required; most municipal supplies were started under private ownership. Private ownership is able to extract larger earnings during the lean years by sharper insistence on efficiency of employees than is possible in a municipality. Municipal ownership means more employees and lessened impetus to attain results. On the other hand, municipal ownership eliminates items of overhead expense, including municipal, state, and federal taxes and the dividend requirement; some plants under municipal ownership have reduced rates. Private companies are subject for local, state and federal taxes.

State Regulation. Water-supply companies or water departments of municipalities are required, in most states, to submit plans for approval both to the state department of health and to the state utility commission. In some states having water supply commissions, they also pass on the plans. Operation of plant, particularly treatment plants, is subject to state supervision. In most states, the utility commissions have regulations covering meter testing, meter reading, accounting, rates, etc.

Appraisal of Plants.† In appraising water companies, the going-concern value should be considered, based on the fact that the plants are in actual con-

* Termed "compound interest" method.

† The reader is referred to following in *Trans Am Soc C E*: "The Going Value of Waterworks," Vol 73, 1911, "The Valuation of Public Service Corporation Property," Vol 72, 1911; "Valuation of Waterworks Property," Vol 38, 1897; "Valuation and Fair Rates in the Light of the Maine Supreme Court decisions in the Waterville and Brunswick Cases," Vol 64, 1909; Report of Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and other Public Utilities," Vol 81, 1917. Also report of Committee on Valuation to Am. Elec. Ry. Assn., 1919; and to the following books: Grunsky, "Valuation, Depreciation and the Rate Base" Wiley, 1918; Raymond, "What Is Fair," Wiley, 1914, Maltbie, "Theory and Practice of Public Utility Valuation," McGraw-Hill Book Company, Inc, 1924; Riggs, "Depreciation in Public Utility Properties and its Relation to Fair Values," McGraw-Hill Book Company, Inc. 1922.

nection with paying customers, and that, if purchased, full income would start from day of purchase, whereas a reproduction of the existing plant, while costing the same, would have to wait one or more years for a normal demand for water.

Cost-of-reproduction valuation is arrived at, in general, by establishing the cost of reproduction, including a percentage for overhead costs,* deducting depreciation, and adding going-concern value. "Reproduction-new" theory of value has been decided by the U. S. Supreme Court[11] to be proper basis for public utility rate making; this reaffirms the famous Smyth-Ames decision of Justice Harlan; the minority report dissents from this basis (see Metcalf in *J. A. W. W. A.*, Vol. 11, 1924, p. 1).

Object of appraisal affects the valuation. A valuation for rate making might fairly have a different basis from one for condemnation; one for selling would also result in different figures from either of the others. In considering a valuation report, it is important to know its object (see "Purposes Should Govern Waterworks Valuations," by Ledoux, *E. N. R.*, Oct. 4, 1917, p. 633). Where a municipality desires to acquire a utility, it has the option of negotiation with the owners, of exercising its right of eminent domain, or of building its own plant. The last possibility is often used as a basis for fixing the condemnation cost of the private plant; obviously, the municipality should not pay more than the value of a new plant, depreciated to the age of the system which is to be acquired.

Contentious items are those of depreciation, franchise value, and going-concern value. If the franchise is not exclusive, there is no legal bar to another company or the municipality installing paralleling lines; in the condemnation proceedings of the City of New York[12] against the Citizens' Water Supply Co., the company claimed a franchise value of $1,250,000, the city allowed nothing, and the Condemnation Commission allowed $1, on the above basis.

Going-concern value should cover development costs, consolidation costs, superseded property, and created value.

The Committee on Valuation of Am. Elec. Ry. Assn., says:

Depreciation has been classed by all students of the subject as an operating charge and not a capital account item . . . The only provision for taking care of depreciation is by making proper repairs and proper renewals where necessary . . . The full original investment remains in the property and the investor is entitled to a return upon every dollar until the investment is repaid to him . . . There is a certain amount of total accrued depreciation that never can be taken care of in any operating, growing property . . . When a property has been maintained in good operating condition, no deduction should be made for accrued depreciation in an appraisal to determine investment value for rate making, or for sale to municipality.[13]

Fair return is defined by Committee on Valuation of Am. Elec. Ry. Assn.[13] as:

An investor in a company is entitled to a reasonable return on his actual original investment, plus the appreciation of the property, including its value as a going concern, as compensation for his "initial risk" or "hazard" and his skill in successfully operating the property.

Ledoux estimates this at 63 per cent.[10]

Table 308. Cost of Unskilled Labor and Materials in Waterworks in the United States, 1915–21*

Metcalf[15]

Item	Number years	Prices per unit 1915	1916	1917	1918	1919	1920	1921	Per cent. increase over 1915, (prewar basis) 1916	1917	1918	1919	1920	1921	1923
Unskilled labor, cents per hour:															
(a) Eastern Group	15–18	23.0	26.7	30.4	40.2	43.8	52.5	43.4	16	32	75	91	128	87	124
(b) Central Group	17–12	21.7	25.3	26.9	37.2	36.8	47.1	40.9	17	24	71	70	117	88	100
(c) Southern Group	6–12	17.9	20.6	24.5	34.3	34.4	42.1	35.7	15	37	92	92	135	99	102
(d) Western Group	7–8	27.0	28.5	31.4	41.8	46.4	54.7	53.5	5	16	55	72	105	98	87
(e) Average	......	22.4	25.3	28.3	38.4	40.4	50.0	43.5	13	27	71	80	123	94	103
Cast-iron pipe per 2000 lb., approx	17–44	$24.23	$30.70	$51.60	$67.74	$69.20	‡76.53	52.30	27	113	179	184	216	116	134
6-in. valves	11–40	11.18	12.64	19.13	19.13	20.73	24.72	21.93	13	71	71	85	121	96	111
12-in. valves	3–38	34.78	41.53	65.22	65.02	59.66	68.90	62.13	19	88	87	72	98	79	87
2-way hydrants	6–38	26.69	32.04	43.13	51.80	47.16	52.30	47.10	20	62	94	77	96	77	136
Coal per 2000 lb.:															
(a) Eastern Group	13–15	$ 2.98	$ 3.80	$ 5.96	$ 6.00	$ 5.41	7.34	5.81	27	100	101	82	146	95	102
(b) Central Group	8–12	2.41	2.77	3.75	4.53	4.55	5.84	5.69	15	56	88	89	142	136	93
(c) Southern Group	7–12	1.92	2.01	3.03	3.89	3.78	5.35	4.45	5	58	102	97	179	132	68
(d) Western Group†	5–4	3.97	4.37	6.31	7.92	4.70	5.60	5.29	10	59	99	18†	41	33	1
(e) Average of Groups	......	$ 2.82	$ 3.24	$ 4.77	$ 5.57	$ 4.61	$ 6.22	$ 5.43	15	69	97	63	121	92	65
Fuel oil, cents per gal.:															
South	1–3	1.80	1.80	2.00	4.28	......	2.35	2.93	0	11	138	......	30	62	46
West	1–4	1.38	1.50	2.57	4.05	4.09	4.16	4.16	9	86	193	197	201	201	60
Alum, per lb.:															
(a) Eastern Group	......	1.12	1.72	1.48	1.45	1.66	1.90	1.75	54	33	29	48	69	57	38
(b) Central Group	......	.91	.91	1.25	1.50	1.40	2.66	1.86	0	37	65	54	192	104	78
(c) Southern Group	......	1.08	1.38	1.48	1.78	1.56	2.17	1.81	28	37	65	44	101	67	56
(d) Western Group	......	1.14	1.21	1.51	1.53	1.79	2.04	2.23	6	32	34	57	79	96	115
(e) Average	......	1.06	1.30	1.43	1.56	1.60	2.21	1.84	22	35	47	51	108	73	72

* See also Metcalf in *J. A. W. W. A.*, Vol. 13, 1925, p. 375.
† Small number makes record of doubtful value.
‡ Range $50.60 to $83.50 per ton.

Waterworks Bonds. Sherman[14] argues that 30 years is the average remaining life in waterworks plants, and that bonds secured by such property should be limited to this term. The term of bonds on municipally owned works is fixed by law at 5 years in Massachusetts and 30 years in New Jersey. The Pierson Act* of New Jersey (1916) also controls the rate of amortization by stipulating that the amortization in any one year shall not exceed by more than 50 per cent. that of any preceding year.

From records of a number of plants Sherman[14] concludes that the average depreciation is 20 per cent. and that 80 per cent. of the value still exists. He would, therefore, issue bonds up to 80 per cent. of the cost of the works. The financing of municipally owned works should be on same basis as private corporations, that is, self-supporting. Bonds for large works should be sold serially to cut down interest charges during construction (for tabulation of interest charges during construction of large dams, see *Elect. World*, July 17, 1920, p. 117).

Costs of waterworks vary widely, and are influenced by geographical location, topography, geology, and many other conditions. Table 308 is useful in revising prewar costs to date. An interesting example of cost segregation on small works is Harroun's paper on waterworks for Porterville, Cal. (pop. 2000) (*T. A. S. C. E.*, Vol. 54, 1905, p. 235).

Cost of supplying water varied in the United States in 1904 from $20 per million gal. in some large cities to $300 in small, unfavorably situated plants; the average of 22 cities, none larger than Cleveland, was $92, Cleveland being $23.† Chicago is estimated at $19.[16] Costs, exclusive of capital charges, increased in Reading,[17] Pa., from $15.62 per million gal. in 1913–1914 to $32.82 in 1921. Pumping costs rose from $5.97 to $15.50.‡ Capital charges at Columbus in 1922–1923 amounted to over 35 per cent. of total operating cost: water treatment 23 per cent., and pumping 11 to 14 per cent.[18] A table in *E. C.*, June 8, 1921, p. 876, compares 1913 and 1917 costs in 29 American cities (see particularly Hazen, "Meter Rates for Water Works" (Wiley, 1916).

Water-main and service extensions are paid for by the municipality, by the water taker, or prorated. For symposium on practice *re* services, see *J. N. E. W. W. A.*, Vol. 38, 1924, p. 135, and "Financing Water-main Extensions," by Blomquist, *J. A. W. W. A.*, Vol. 11, 1923, p. 789. Cincinnati and Philadelphia charge $2 per front ft. of abutting property; some cities charge as low as $1, etc. (see Jordan, *J. A. W. W. A.*, Vol. 12, 1924, p. 94).

Rates§ are practically the only revenue derived from the water system, and must yield sufficient revenue to meet the following yearly charges: interest on outstanding bonds; amortization payments into the sinking fund to retire bonds; operating expenses, including repairs and depreciation reserves for replacing valueless equipment. Costs of extensions are sometimes taken from yearly revenues, and sometimes assessed on the property asking for such

* Sect. 3, Ch. 252, Acts of 140th Legislature, N. J., 1916, p. 527 "All bonds hereafter issued ... mature in not exceeding 50 years and in annual installments commencing not more than 2 ... from their date and no installment shall be more than 50 per cent. in excess of the amount of ... smallest prior installment.

† Total maintenance plus interest on bonds.

‡ For Chicago costs, see p 474.

§ See particularly "Meter Rates for Water Works," by Hazen (Wiley, 1916), and "Rates for W... Supply," by Wagner, *J. N. E. W. W. A*, Vol 29, 1915, p. 1.

Table 309. Cost of Cubic Feet of Water at Stated Rates per 1000 Gals.

No. of cu. ft.	Cost per 1,000 Gals. (For cost per mg. move decimal point 3 places to right)							
	5 Cts.	6 Cts.	8 Cts.	10 Cts.	15 Cts.	20 Cts.	25 Cts.	30 Cts.
20	$0.007	$0.009	$0.012	$0 015	$0.021	$0.030	$0.037	$0.045
40	0.015	0.018	0.024	0 030	0.045	0.060	0.075	0.090
60	0.022	0.027	0.036	0 045	0.066	0.090	0.112	0.135
80	0.030	0.036	0.048	0.060	0.090	0.120	0.150	0.180
100	0.037	0.049	0.060	0.075	0.111	0.150	0.187	0.224
200	0.075	0.090	0.120	0.150	0.225	0.299	0.374	0 449
300	0.112	0.135	0.180	0.224	0.336	0.449	0.561	0.673
400	0.150	0.180	0.239	0.299	0.450	0.598	0.748	0 898
500	0.188	0.224	0.299	0.374	0.564	0.748	0.935	1.122
600	0.224	0.269	0.359	0.449	0.674	0.898	1.122	1.346
700	0.262	0.314	0.419	0.524	0.786	1.047	1.309	1.571
800	0.299	0.350	0.479	0.598	0.897	1.197	1.496	1.795
900	0.337	0.404	0.539	0.673	1.011	1.346	1.683	2.020
1,000	0.374	0.449	0.598	0.748	1.122	1.496	1.870	2 244
2,000	0.748	0.898	1.197	1.496	2.244	2.992	3.740	4 488
3,000	1.122	1.346	1.795	2.244	3.366	4.488	5.610	6 732
4,000	1.496	1.795	2.393	2.992	4.488	5.984	7.480	8 976
5,000	1.870	2.244	2.992	3 740	5 610	7.480	9.350	11 220
6,000	2.244	2.692	3.590	4 488	6.732	8.976	11.220	13 464
7,000	2.618	3.141	4.189	5.236	7.854	10.472	13.090	15 708
8,000	2.992	3.590	4.787	5.984	8.976	11.968	14.961	17 953
9,000	3.366	4.039	5 385	6.732	10.098	13.464	16.831	20 197
10,000	3.74	4.488	5.984	7.480	11.122	14.961	18.701	22 441
20,000	7.48	8.976	11.968	14.961	22.443	29 992	37.402	44 882
30,000	11.22	13.46	17.95	22.44	33.664	44.88	56.10	67 32
40,000	14.96	17.95	23.94	29.92	44.885	59.84	74.80	89 77
50,000	18.70	22.44	29 92	37 40	56.103	74.80	93.50	112 20
60,000	22.44	26.92	35 90	44 88	67 323	89.76	112 20	134 64
70,000	26.18	31.41	41 89	52 36	78 543	104 72	130 90	157 08
80,000	29.92	35.90	47.87	59.84	89.766	119.68	149.61	179.53
90,000	33.66	40.39	53 85	67 32	100 986	134 64	168.31	201 97
100,000	37.40	44 88	59 84	74 80	111 22	149 61	187.01	224.41
200,000	74 81	89 76	119 68	149 61	224 43	299.22	374.02	448 82
300,000	112 20	134 64	179 53	224 41	336.63	448 83	561.03	673 24
400,000	149.61	179.53	239.37	299.22	448.85	598.44	748.05	897.66
500,000	187.01	224 41	299 22	374 02	561.03	748.05	935.06	1,122.07
600,000	224.41	269 29	359.06	448.82	673 23	897.66	1,122.07	1,346 49
700,000	261.81	314 18	418 90	523 63	785 43	1,047 27	1,309.08	1,570.88
800,000	299 22	359 06	478 75	598 44	897 66	1,196 88	1,496.10	1,795 32
900,000	336 62	403 94	538 59	673 24	1,009 86	1,346 49	1,683.11	2,019 73
1,000,000	374 02	448 83	598 44	748 05	1,122 06	1,498 10	1,870 12	2,244.15

extensions.* In addition to these charges, private water companies must meet taxes, dividends, and the interest on investment. This last item also exists under municipal ownership, but is not considered a charge against the system. Rates must also cover cost of providing water diverted to public uses—fires, street cleaning and sprinkling, sewer flushing, horse troughs, fountains, unmetered public parks and buildings—as well as loss by leakage.

Rates must also be adjusted to distribute the surcharges for fire protection; commonly, the municipality pays a unit charge per hydrant in service, or a per capita tax is levied. Fixing of rates is vested in various state regulatory commissions. Saville[19] cites as method most approved by public utility commissions in rate-making decisions a composite charge consisting of a unit charge per hydrant for maintenance and operation, plus another unit charge for pipe capacity and other costs of excess service per lin. ft. of pipe in service. See epitome of current practice by Burnham, *J. N. E. W. W. A.*, Vol., 38, 1924, p. 111, and Jordan, *J. A. W. W. A.*, Vol 12, 1924, p. 95.

* See "Construction of Rate Schedules," by Ehlers, Pa. W. W. A, 1923, p 191.

**Table 310. Meter Rates in Selected American Cities,* 1920 [20]
(Municipal operation)**

City	Population (nearest 1000)	Per cent. metered	Meter rates, cents: Highest domestic, cts per 1000 gals.	Lowest commercial, cts. per 1000 gals.	Min. annual charge, dollars
Mobile, Ala.	69	56	15	10	6
Prescott, Ariz.	8	100	100	25	24
Blytheville, Ark.	8	100	40	10	18
San Diego, Cal.	75	100	14.7	14.7	....
Colorado Spring, Col.	30	2	15	8	12
Hartford, Conn.	138	98	16	8	5
Wilmington, Del.	110	86	10	7.3	10
Daytona, Fla.	5	100	15	....	5
Augusta, Ga.	53	100	25	12.5	9
Boise, Ida.	28	87	27	12.5	12
Peoria, Ill.	76	60	30	3	3
Fort Wayne, Ind.	86	92	16	6.5	6
Council Bluff, Ia.	36	100	35	10	6
Chanute, Kan.	10	100	30	10	3
Ashland, Ky.	15	54	35	10	12
New Orleans, La.	387	100	10	7	3
Waterville, Me.	13	2	25	25	40
Hagerstown, Md.	28	93	30	8	6
Cambridge, Mass.	110	47	10	10	5
Jackson, Mich.	48	100	13.3	10	7
Duluth, Minn.	99	78	20	10.7	6
New Albany, Miss.	3	88	37.5	25	12
Poplar Bluff, Mo.	8	6	25	10	6
Bozeman, Mont.	8	3	100	10.7	
Grand Island, Neb.	14	100	16	5	6
Elko, Nev.	2	56	30	7.5	....
Concord, N. H.	22	67	22.2	5	10
Kearny, N. J.	27	100	20	16	7
Carlsbad, N. M.	2.5	96	30	10	24
Johnstown, N. Y.	11	9	40	6.6	9
Wilmington, N. C.	33	40	21.6	11.5	13
Carrington, N. D.	1.5	100	66.7	33.3	6
Athens, O.	6	29	20	9	7
Blackwell, Okla.	12	99	40	6.7	9
Portland, Ore.	258	31	10.7	8	6
Harrisburg, Pa.	76	71	5.7	5.7	4
Providence, R. I.	236	94	20	10	8
Charleston, S. C.	68	98	24.7	6	12
Watertown, S. D.	10	29	25	10	8
Memphis, Tenn.	162	79	33.3	12	12
Port Arthur, Tex.	22	100	30	13	6
Salt Lake City, Utah	118	32	7.3	6	6
Burlington, Vt.	23	100	20	8	6
Alexandria, Va.	18	39	30	8	12
Seattle, Wash.	316	100	13.3	5.3	6
Wheeling, W. Va.	54	9	15	5	....
Madison, Wis.	38	99	10	5.3	4
Pine Bluffs, Wyo.	0.9	100	30	10	None

* See also Jordan's table in *J. A. W. W. A.*, Vol. 12, 1924, p. 88.

For unmetered services,[21] the flat rate is fixed by one of the following: (*a*) frontage on the main with an excess charge for fixtures beyond the stipulated number, as used in New York and Chicago; (*b*) number of rooms, with additional charges where fixtures exceed the schedule; this method is in

wide use; (c) a flat minimum rate, based on size of supply tap, with charges for each fixture; Philadelphia has this.

Meter Rates.[21] (a) fixed rate per 1000 gal.; (b) sliding scale per 1000 gal.; with maximum rate for minimum use, (c) fixed charge per annum, covering a stipulated consumption, with excess use paid for at a fixed or sliding rate; (d) a fixed charge per annum to cover "readiness to serve" with a fixed rate per 1000 gal. for water used; (e) minimum rate plus a service charge plus sliding or fixed rate per 1000 gal

Sinking-fund Reserve to Cover Depreciation or Amortization Charges.[24]

$$x = \frac{r}{(1 + r)^n - 1},$$

where x = annual payment made at end of each year to accumulate $1 at the end of n years, the instalments bearing interest at r per cent, compounded annually. Figure 338 is plotted from this formula.

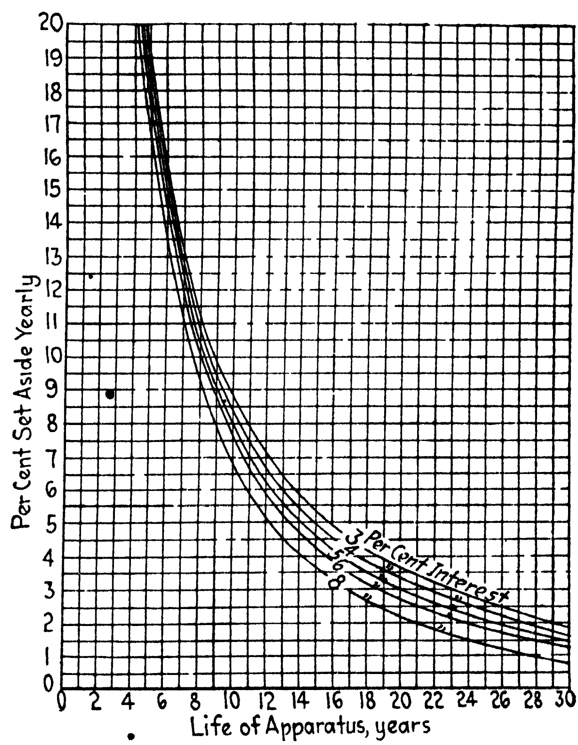

FIG. 338.—Annuities for sinking fund to cover depreciation or amortization charges for different terms of life and rates of interest [24]

Equal Payments to Liquidate a Debt* with Interest.[25]

$$a = \frac{B(1 + r)^n r}{(1 + r)^n - 1},$$

where a is the amount of each of the equal payments; B is the total principal or bond issue; r, the interest rate for the fixed equal periods between pay-

* See also Shepard, *E N R*, May 2, 1918, p 871.

ments; n is the number of payments.* The total amount to be paid, including interest, is na. For methods of calculating average rate of interest for stipulated values of a and n, see Sullivan, *E. N. R.*, Jan. 29, 1925, p. 204.

Comparison of Ultimate Economy of Materials of Differing Durabilities.[30] †

Let y = First cost of less durable article

x = First cost of more durable article to be on an ultimate equality with y

N = Number of years' duration of x

n = Number of years' duration of y

i = Rate of interest per annum compounded annually in decimals of 100; for example, 5 per cent = .05

$$x = y \frac{(1+i)^N - 1}{(1+i)^N - (1+i)^{N-n}}$$

Table 311. Probable Life of Waterworks Elements[26]

Committee on Depreciation of the A. W. W. A.

	Life in Years
Large storage reservoirs, well located	75 to 150
Heavy earth or masonry dams	75 to 150
Large masonry conduits and tunnels	75 to 150
Cast-iron pipe, large	75 to 125
Cast-iron pipe, small	30 to 70‡
Wrought-iron or steel pipe, large	30 to 75
Wrought-iron or steel pipe, small	25 to 40
Wood-stave pipe, large	30 to 60
Services, wrought-iron and steel	15 to 30
Services, wrought-iron and lead	40 to 80
Small distribution reservoirs	50 to 75
Standpipes, wrought-iron and steel	30 to 60
Standpipes, reinforced-concrete	50 to 60
Valves	40 to 60
Hydrants	30 to 50
Meters	20 to 30
Pumping machinery, high-duty large units	35 to 60
Pumping machinery, high-duty small units	25 to 50
Pumping machinery, ordinary direct-action	25 to 50
Pumping machinery, centrifugal, not geared	20 to 30
Pumping machinery, centrifugal, geared	15 to 25
Steam engines	20 to 40
Boilers	15 to 30
Electric generators and motors	20 to 30
Filter plants, masonry	30 to 50
Filter plants, wood	15 to 30
Buildings, masonry	30 to 60
Buildings, wood	20 to 40
Stacks, masonry	25 to 50
Stacks, steel	10 to 25

* If period = 30 days and annual rate = 6 per cent, r becomes $0.06 \times \frac{30}{365} = 0.005$.
† Of value in comparing steel, wood-stave reinforced concrete and cast-iron conduits.
‡ In slow-growing and small cities, 50 to 90 years

Alvord and Burdick[27] for 54-in. conduit at Iron Mountain, Mich., estimated riveted steel at 35 years, galvanized iron, 15; wood stave, 10; and wooden flume, 20. New York State law for bond issues fixes life of water-works at 25 years.[26]

THREE-PLACE LOGARITHMIC TABLES*

Table 312. Logarithms of Numbers

N	Log	N.	Log	N	Log	N	Log	N.	Log.	N	Log.
20	301	50	699	80	903	110	041	140	146	170	230
21	322	51	708	81	908	111	045	141	149	171	233
22	342	52	716	82	914	112	049	142	152	172	236
23	362	53	724	83	919	113	053	143	155	173	238
24	380	54	732	84	924	114	057	144	158	174	241
25	398	55	740	85	929	115	061	145	161	175	243
26	415	56	748	86	934	116	064	146	164	176	246
27	431	57	756	87	940	117	068	147	167	177	248
28	447	58	763	88	944	118	072	148	170	178	250
29	462	59	771	89	949	119	076	149	173	179	253
30	477	60	778	90	954	120	079	150	176	180	255
31	491	61	785	91	959	121	083	151	179	181	258
32	505	62	792	92	964	122	086	152	182	182	260
33	519	63	799	93	968	123	090	153	185	183	262
34	531	64	806	94	973	124	093	154	188	184	265
35	544	65	813	95	978	125	097	155	190	183	267
36	556	66	820	96	982	126	100	156	193	186	270
37	568	67	826	97	987	127	104	157	196	187	272
38	580	68	833	98	991	128	107	158	199	188	274
39	591	69	839	99	996	129	111	159	201	189	276
40	602	70	845	100	000	130	114	160	204	190	279
41	613	71	851	101	004	131	117	161	207	191	281
42	623	72	857	102	009	132	121	162	210	192	283
43	633	73	863	103	013	133	124	163	212	193	286
44	643	74	869	104	017	134	127	164	215	194	288
45	653	75	875	105	021	135	130	165	217	195	290
46	663	76	881	106	025	136	134	166	220	196	292
47	672	77	886	107	029	137	137	167	223	197	294
48	681	78	892	108	033	138	140	168	225	198	297
49	690	79	898	109	037	139	143	169	228	199	299
50	699	80	903	110	041	140	146	170	230	200	301

* John Wiley & Sons, 1900.

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INDEX

A

Acids, formulas, 676
  ionization, 650
  solutions, pH concentration, 653
  vegetable, weak, 654
Acre-feet, equivalents, 816, 818
Adelaide, rainfall, 23
Adsorption, 650, 652
Aeration, 645–649
  coke trays, 648
  deep reservoirs, 636
  iron removed, 699
  odors removed, 645
  soil, 628
  spray, 646, 648
Aerator box, 646
Aerators, 645
Agar, bacteriological medium, 600
Agitator, Dorr, 660
Agriculture, water consumption, 575
Air, chamber, pumping engines, 474
    water hammer, 782
  flow, 772
  gases, water absorbs, 584
  intakes, exclusion, 81
  lift pump, (see below).
  pumps, boilers, 510
  solubility, 583
  valves, 446–448
    Crispin, 447
    design, 446
    discharge, 448
    flow of air, 772
    negative-pressure, 447
    use, 446
    vaults, 359, 447
    wood-stave pipe, 372
  in water, 779
  weight, 507, 584
Air-lift pump, 249–259
  advantages, 254
  booster system, 258
  calculations, 252
  defined, 249
  disadvantages, 253
  efficiency, 255
  Frizell system, 250
  Ingersoll-Rand formula, 252
  nomenclature, 249
  piping, types, 251
  Pohle's system, 250, 251
  principle, 249
  return air type, 253
  Saunders, 251
  terminology, 249
  tests, 255, 256
  types, 250
  use, 249, 255
Akron, filters, 722
  steel pipe corrosion, 325
Albany, filters, 739
  rainfall and temperature, 12
  subsiding basin, 642
  water analysis, 620
Albuminoid ammonia, water, safe limits, 585
    significance, 585
    tests, 595
Alcement, 794
Algae, canal hydraulics, 272, 297
  Connecticut waters, 609
  covered reservoirs, 709
  distribution reservoirs, 538
  growth, 638
    storage effect, 633
Algae, stumps in reservoir, 110
  in well pipes, 222
Algicides, 665
Alkali, cast-iron pipes, 387
  soils, concrete deteriorates, 373, 375, 797
  solutions, pH concentration, 653
Alkalinity of water, corrosion relation, 810
  hardness measures, 586
  significance, 586
  test, 598
    precision, 738
  theoretical, 654
  well water supply, 651
Allegheny river, sediment, 63
Altitude, control valves, 451, 536
  pumping, 472
  rainfall effect, 4
Alum (see *Alumina, sulfate*).
Alumina sulfate, 653
  acid, 653
  alkalinity, theoretical, 654
  bauxite, 653
  chlorine aids, 654
  color removal, 657
  concrete affected, 801
  corrosive, 669
  costs, 658
    range, 831
  dry feed, 670
  feeding devices, 666–676
    small filters, 719
  formulas, 676
  package, 677
  quantities, 656, 657
  reaction, 654
  solubility, 664
  solutions, floc formation, 651
  tanks, 669
  volume-weight, 678
Aluminum, atomic weight, 678
  -hydrate, slow filters, 745
  sulfate, (see *Alumina*).
Ambursen dams, 145
  principle, 116
American cities, water consumption, 576
A. S. M. E., boiler code, 508
A. W. W. A., cast-iron pipe specifications,
    381, 389
  hydrant specifications, 455
  meter specifications, 465
  pipe freezing, report, 421
  service standards, 432
  valve standards, 436
Ammonia, free, (see *Nitrogen*).
Ammonium, salts, formula, 676
    solubility, 664
Amortization, sinking fund reserves, 835
Anabaena, growth, 639
  in ice, 613
  odors, 613, 614
Anchor ice, 84
Anchorage, bends, c. i. pipe, 404
  steel pipe, 330, 334
Animals, burrowing, earth dams, 193, 196, 208
Anions, 653
Annicut, 150
Annuities, sinking fund, 835
Anthracite soal, defined, 513
  evaporative power, 507
  sizes and analyses, 514
Aphanizomenon, odor, 614
Apishapa dam, failure, 203
  statistics, 219
Appomattox river, run-off, 59
Appraisals, water-works, 829

Aprons, overflow dams, 147, 152, 153
rock-fill dam, 190
Apulian aqueduct, bridges, 288
reserve storage, 535
water consumption, 571
Aquaphone, 425
Aquatic growths, canals, 297
Aqueducts (masonry), 280–300
appurtenances, 297
bridges, 289
Apulian aqueduct, 288
capacity, 297, 767
loss, 283
required, 280
chambers, floor covers, 300
gate-houses, 529
plug valves, 297
cleaning, 297
concrete-lined, capacity, 297, 767
construction methods, 283
cut-and-cover, 281–289
alinement, 284
Catskill aqueduct, 286
vs. conduit, 283
contraction joints, 287
cracks, 286
cross-section, changes, 284
culverts, 284
curves, 284
economy, 280, 282
expansion joints, 286, 287
forms, 288
foundation embankments, 285
gradients, 281
materials, 283
vs. pipe line, 283
shape, 281, 282
economic gradients, 85, 281
fences, 105
flow, 297, 767
fouling, 296, 375
gate houses, 529
floor covers, 300
plug valves, 297
gradients, 280, 281
growths in, 296, 297, 375
headworks, arrangement, 280
under highways, 284
hydraulic slopes, 281
inverts, 281
life, 836
maintenance, 296
overflow weirs, 281
pressure, gradients, 281
profile, 280
under railroads, 284
screens, 298
spillways, 281
stop planks, 299
terminology, 280
tunnels, 290–296
capacity, unlined, 296
excavation, 290
grade, 280, 281, 297
leakage, 294
vs. pipe lines, 280
pressure, 291–295
progress, 290
shafts, 291
shapes, 290
unwatering, 297
types, 280
use, 280
water temperature, 288
Aquifer, defined, 71
Arbor vitae, forestry, 111
Arch, groined, 544–547
multiple-arch dam, 143
Arched dams, 135–141
adaptability, 135
advantages, 135
constant-angle, 137, 141
cracks, 135
cylinder formula, 136
design formulas, 136
disadvantages, 135
economy, lack of, 117
Arched dams, existing, data, 139
gravity type, 116
multiple-arch, 142–145
principle, 116
sites, 135
thermophones, 123
uplift, 135
Area, drainage, defined, 87
Armco pipes, 338
Arrowrock dam, 167
concreting rate, 159
contraction joints, 123
gate friction tests, 806
outlets, tar coating, 798
velocity, 105
sand cement, 794
section, 176
thermophones, 123
uplift, 119
Arsenic, atomic weight, 678
Artesian well, 75
jetting, 228
Ashokan dam, composite type, 116
construction, 160
contraction joints, 123
data, 163, 176
ice thrust, 118
masonry, 802
statistics, 218
uplift, 119, 163
Ashokan dikes, core wall, 198
paving, 199
rolling, 207
slips, 202
statistics, 219
Ashokan reservoir, cemeteries, 104
clearing and grubbing, 108
core walls, forms, 159
stumps, 109
wave height, 199
Asphalt coating, c. i. pipe, 388
steel pipes, 318, 322
Assabet bridge, 289
Assiut regulator, 153
Assoc. Factory Fire Ins. Cos., centrifugal pumps, 484
standpipe specifications, 562
valve standards, 436
Assuan dam, data, 163
Asterionella, ground waters, 639
odor, 614
Astoria, rainfall, 5
Atlanta, rainfall and temperature, 15
Atlantic City, high-pressure fire system, 402
Atmosphere, moisture capacity, 27
one, equivalent, 817
pressure conversion, 818
Attleboro standpipe, 550, 551, 552, 553, 554, 556
Auger boring, wells, 231
Augusta, Ga., rainfall, 5
Austin, Pa. dam, failure, 125
Austin, Tex., dam, failure, 119, 125, 145
gates, 104
statistics, 161, 168
reservoir silt, 107
Automobile camps, 625

B

Bacillus (see also *Bacterio*), **acid-formers**, 589
enteriditis, 589
paratyphosus, 589
typhosus, ice kills, 613
longevity, 611
tests, 589
vitality, 637, 638
Welchii, 589
Backfill, c. i. pipe, 404
Backwater, damage, 93
Bacteria coli, copper sulfate kills, 665
count, yearly range, 611
ground waters, 605
index, 589 616
presence, significance, 591
sanitary importance, 605
temperature effect, 611

Bacteria coli, test, confirmed, 589
    positive, 611
    precision, 738
    presumptive, 589, 593
    value, 589
  vitality, 637, 638
  well water, 604
Bacteria, denitrifying, 603
  filtration removes, 702
  ground waters, 602, 603
  iron, 588
    corrosion, 387
  longevity, 626, 637
  Ohio River, 616
  origin, 637
  precipitation, 650
  removal, colored waters, 657
    rapid filters, 729
    turbid waters, 657
  significance, 589
  size, 702
  soil, 603
  species work, 589
  surface waters, 610
  test, precision, 738
Bacteriology, of water, 588
Baden-Baden, springs, 268
Baffles, contact, 695
  mixing coagulants, 659
  reservoir, 536
  sedimentation basins, 644
Baldwin reservoir, construction plant, 541
Balmorhea, dam, construction, 208
  reservoir, paving, 201
Balsam, forestry, 111
Baltimore, filters, 743
  high-pressure fire system, 402
  Loch Raven reservoir, 107
  rainfall, 5, 14
  subsiding basin, 642
  temperature, 14
Barium, atomic weight, 678
  carbonate, coagulant, 655
    formula, 677
    water softening, 685, 689, 693, 694
  salts, poisonous, 685
Barker dam, data, 163
Barnaul, rainfall, 23
Barometer, heights, pressures, 472
  records, U S, 826
Barossa dam, data, 139, 163, 177, 187
Basins, cleaning, 640
  mixing, 659, 660
  sedimentation, 536
  subsidence, 640-644
Bassano dam, composite type, 116
  concreting in winter, 795
  construction, 160, 207
  gates, frost protection, 455
  movable crest, 104
Basswood, forestry, 111
Bathing, in reservoirs, rules, 626
Baths, public, water consumption, 571
Bauxite, 653
  volume-weight, 678
Bazin, weir formula, 785, 791
Bear Valley dam, data, 139, 163, 178
Bearings, centrifugal pumps, 486
Beech, ornamental planting, 111
Beetaloo dam, data, 163, 178
Belabula dam, 141
Belfast (Me.), filters, 744
Bell-and-spigot pipe, 389, 408, 412, (see *Cast-iron pipe*).
Belle Fourche dam, paving, 200
  statistics, 218, 219
Belt drive, pumps, 520
Bends, cast-iron pipe, 407-411
    quarter, standard flanged, 345
  pipe, combined, 408-411
  steel pipe, 330
Beowaine, Nev., rainfall, 5
Berlin, rainfall, 23
Berm, earth dam, 203
Bermuda grass, embankments, 112
Bernoulli's theorem, 651
Betwa dam, data, 161, 168
Bibliography (see end of each chapter).
Big Meadows dam, statistics, 218
Birch, white, ornamental planting, 111
Bismarck, rainfall, 5
Bitose, 315
Bitumastic enamel, 317, 388
Bituminous coal, evaporative power, 507
Black Canyon dam, concrete proportions, 795
  sluice gates, 454
  type, 146
Bleaching powder, 662
  disinfectant, 632
  feeding, 673
  formula, 677
  package, 677
  volume-weight, 678
Blister, rust, 111
Blocks, concrete, masonry dams, 159
Blow-offs, distribution systems, 420
Blue vitriol, 665
Board of Water Supply, (see also *Catskill water supply*)
  clearing and grubbing, 109
Boating, on reservoirs, rules, 626
Bog Brook dam, statistics, 219
Boiler, steam, 506-512
  air pumps, 510
  coals, evaporative power, 507
  combustion, 506
  compounds, 687
  condensers, 510
  corrosion, 681, 687
  evaporation, 506
  feed pumps, piston speed, 480
  feed-water, heating, 509
    troubles, 681, 682
  firing, 508
  foaming, 682
  fuel economizers, 510
  hard waters, 680
  heating surface, units, 507
  horsepower, 506
  life, 836
  makers, 506
  oil burning, 512
  priming, 682
  rating, chimney height, 516
  regulations, 508
  scale formation, 681
  selecting, 508
  stokers, 508
  superheat, 509
  vacuum, 511
Boiling point, water, 507
Bolts, dimensions, 341
  flanged joints, 389
  weight, 344
Bombay, rainfall, 23
Bonds, waterworks, 832
Bonna pipe, 376
Boonton, dam, data, 161, 163, 169, 178
  ice thrust, 118
  masonry, 802
  temperature cracks, 123
  uplift, 119, 163
    reservoir, color reduction, 634
Booster pump, air lift, 258
  distribution system, 470, 529
"Boothal," volume-weight, 678
Boquilla dam, uplift, 120
Borden Brook dam, statistics, 218
Bore-holes, (see *Wells*).
Boring, auger, wells, 231
Boron, atomic weight, 678
Borromite, zeolyte, 687
Boston, evaporation records, 32
  high-pressure fire system, 402
  metering, results, 467
  rainfall, 5, 9
  solid-gate valve, 436
  storage reservoirs, 91
  temperature, 9
Boulders, floods transport, 61
  well sinking, 235
Bouzey dam, failure, 117
Box, weir, 790

Boxes, meter, 467
gate-valve, 440
Boyd's Corners dam, data, 163, 178
Brake horsepower, 471
Brass, cracking, 806
service pipes, 432
Brick chimneys, 517
Bridgeport, storage reservoirs, 91
dam, height raised, 160
Bridges, aqueduct, 289
Apulian, 288
impounding reservoirs, 104
pipe on, 419
leadite, 413
protection, 404
steel pipe, 334, 339
wood-stave pipe, 372
Bridgewater, dam, foundation, 214
pumping, 216
settlement, 214, 215
standpipe, 553, 554
Brisbane Queensland, rainfall, 23
Bromine, atomic weight, 678
Bronze, 805 808
on bronze-friction, 455, 805
corrosion, 807
cracking, 806
manganese, 806
valve stems, 437
Brookline, deferrization plant, 700
infiltration gallery, 260
Brooklyn, infiltration gallery, 261
Brooks, gaging flow, 42
Broths, bacteriological media, 600
Brownian movements, 651
Brule dam, stream control, 158
uplift, 120
Brussels, ground-water yield, 74
infiltration galleries, 260
Buckets, overflow dams, 147, 152, 153
Buffalo, high-pressure fire system, 102
Buildings, cost, 530
on reservoir site, 109
Bump joint, pipes, 355
Burkli-Ziegler, run-off, 61, 95, 284, 285
Burlap covering, steel pipe, 318
Burlington, Vt., rainfall, 5
Burmus Paper Co. dam, 156
Burraga dam, data, 163, 177
Burrator dam, data, 164
Burrin Juck dam, 163, 802
Burrowing animals, earth dams, 193, 196, 208
Bursaria, odor, 614
Butt joints, 302, 309
triple-riveted, 310
Butte City dam, data, 161, 169
Buttress, multiple-arch dam, 143

C

Cabin John Bridge, 289
Cables, electrolysis, 428
well measurements, 242
Calaveras dam, 210, 216, 219
Calcium, atomic weight, 678
hypochlorite, 662
disinfectant, 632
salts, formulas, 676
solubility, 659
Calcutta, India, rainfall, 23
Calexico, evaporation, 31
California, ground water resources, 77
method, well sinking, 223, 225
rainfall, 2, 22, 60
run-off curves, 66
Southern, rainfall and run-off, 60
streams, records, 42
temperatures, 22
Calking, cast-iron pipe, 412
steel pipe, 328
Calyx drill, 229
Cambria dam, statistics, 218
Cambridge, filters, 743
rainfall, 3
Canals (see also *Open channels*)
aquatic growths, 297
sections, 269, 270
Cane Creek, floods, 62
Cantonments, water waste stopped, 423, 424
wood-pipe, 359, 367
Capacity tables, 815
Cape Town, rainfall, 23
Capillary, rise, ground waters, 73
tubes, flow, 756
Carbolic acid, disinfectant, 632
Carbon, atomic weight, 678
dioxide, carbonation, 686
deferrization, 699
in ground waters, 603
solubility, 583
test, precision, 738
water analysis, 598
weight, 584
Carbonation, 686
Carbonic acid, aeration, 646, 647
Cascade aerators, 645
Casings, centrifugal pumps, 485
wells, 224, 226, 233
corrosion, 246
dimensions, 351
safe, 630
sinking, 226, 230
Cast iron, corrosion, 810
Cast-iron pipe, 381 397
advantage, 381
air valves, 446
bedding, 403
bell-and-spigot, 389
advantages, 381
anchorage, curves, 389, 404
bends, anchorage, 389, 404
calking, 412
room, 405
cement joints, 415
cost, 385, 386, 405
range, 831
relative, 358, 359, 369
curves, 407–411
deflection at curves, 408
design of pipe, 384
diameter, non-standard, 381
disadvantages, 381
English standards, 382
expansion joint, 393
factory-made joints, 394
failure, 301
method, 399
settlement, 403
fittings, special, 382
wood pipe, 367
flexible ball joints, 394, 416
foundations, 403, 404
large, dangers, 399
laying, 403 407, 417
cost, 369
length, 389
lead joints, 412, 413, 415
solid joint, 405
lead-hydro-tite, 415
leadite, 413
leakage, 426–428
testing, 411
vs. lock-bar, 381
maintenance, 419
manufacture, 383
manufacturer's standard, 381
plugs, 404, 411
simplex prepared, 305
specials, 382
curve combinations, 409
wood-stave pipe, 367
specifications, 381, 389
testing, 418
after laying, 411
well casing, 234
bends, 409
loss of head, 769
breakages, 385
rocks, 403
on bridges, 404, 413, 419
capacity, 387, 757, 758, 765
age effect, 755, 756, 758
vs. riveted steel, 301, 399, 762
tests, 758, 764
vs. wood, 766

Cast-iron pipe, cement-lined, 374, 388
cleaning, 421
coating, 388
depreciation, 387
connections, 382
corrosion, 387, 810
cutting, 412
de Lavaud, 395
depreciation, 387
depth of laying, 404
design standards, 382
disadvantages, 381
discharge, 387, 757, 758, 765
electrolysis, 428–431
excavation, 406
filling, 412
fittings, flanged standard, 391
flanged, 389
flow (see *Capacity* above).
French pipe, 395
French standards, 382
friction-head, diagrams, 757, 758
table, 755
high-pressure pipe, 393, 394, 396, 403
hi-tensile, 383
hub-and-spigot, 389 (see *bell-and-spigot* above).
in marshes, 388
inspection, foundry, 383
lead-lined, electrolysis, 809
life, 388, 836
new, friction-head tables, 759
nodules, 387
old, friction-head, tables, 759
pressure regulation, 418
protection, 388
salt marshes, 418
sand-spun, 395
sewers, relation, 406
vs. spiral-riveted, 339
vs. steel, 301, 399
stream crossings, 416–419
stresses, refill, 383
superheat, 509
thickness, 382
threaded, 393
trench dimensions, 404, 405
tuberculation, 381, 387, 388, 416, 813
two per street, 406
universal pipe, 395
use, 381
victaulic joint, 394
welding, 346, 347, 420
Castings, steel, vs. c. i., 382
weak, reinforcing, 385
Cataract dam, data, 164
Catawba dam, earth statistics, 219
masonry, data, 161, 169
Catawba river, floods, 62
Catchment area, (see *Watershed*)
sanitary patrol, 625
Catskill Aqueduct, aerator nozzles, 646
air removal, 779
bitumastic enamel, 317
bronze composition, 805
Bryn Mawr siphon, life, 812
burlap coating, 318
capacity, 280
Chezy's C, 272
concrete, experience, 797
mix, 283
cracks, 286
culverts, 285
cut-and-cover types, 281
expansion joints, 287
fences, 105
forms, 288
gate-houses, 529, 530
gate valves, operation, 444
gradients, 281
grouting, 294, 803
Hill View reservoir lining, 540
hydraulic properties, 272, 282
lock-bar pipe, vs. riveted, 765
specifications, 311
manganese bronze, 806
Catskill Aqueduct, mortar lining, pipes, 322
Narrows siphon, coating, 388
leakage, 418
open-cut sections, 286
overflow weirs, 281
pickling steel pipe, 314
pressure tunnels, 291, 295
cement soluble, 794
reserve storage, 535
reservoirs, use, regulations, 626
screens, 299
siphons, flow, 769
slopes, 281
steel pipe, cover, earth, 330
design, 307, 313
lining, 323
rivet size, 302
riveted joints, design, 306–310
riveting, specs., 302
siphons, 323
steel plate, 310
tunnel, leakage, 296
tunneling rates, 290
valve sizes, 437
valve stems, bronze, 437
Venturi meters, 463
Catskill Water Supply, (see also above, and *Ashokan, Kensico, Schoharie*).
clearing and grubbing, 109
grassing specifications, 113
grouting, 127
reforestation, 111
Caustic soda, formula, 677
package, 677
water softening, 684
Cedar, red, ornamental planting, 111
white, forestry, 111
Cedar Grove dam, statistics, 220
Cedar River reservoir, leakage, 108
Cellular dams, 141–146
Cement, 793
alumina, 794
gun, lining pipe, 323
lined pipe, 374, 388
capacity, 297
electrolysis, 429
services, 374, 432
pipe, 373
pipe joints, 415
vs metalium, 414
solubility, 794
tufa, 794
Cemeteries, distribution system, 407
reservoirs, 104
Centrifugal, c. i. pipe, 395
concrete pipe, 377
Centrifugal pumps, 482–497
adaptability, 482
advantages, 484
air troubles, 488
bearings, 486
bed plates, 486
brake horsepower, 494
capacity, curves, 495, 496
relations, 489
casings, 485
characteristics, 494
check valves protect, 451
defined, 471
designing data, 487
Diesel engines, 483
dimensions, 489–491
disadvantages, 484
dry wells, 484
efficiency, 494, 495
calculations, 494
vs. other pumps, 495
electric drive, 487, 520, 522
adaptability, 482
characteristic curves, 495
motor proportions, 524
engine drive, 488
fire-pump design, 495
fire-service, 485
fittings, 487
floor space, 489–491
foot valves, 489

Centrifugal pumps, foundations, 486
gas engines, 483
head, relations, 489
horizontal vs vertical, 484
horsepower diagram, 492
impellers, 485
life, 836
noise, 489
operation, 488
series and parallel, 484
performance, 477, 497
power relations 489
pressure relations, 489
prime movers, 487
priming, 489
troubles, 484
principle, 482
vs reciprocating, 476, 484
shafts, 485
specifications, 483
speed, chart, 493
correct, 482
excessive, 498
relations, 489
specific, 489
stages, 484
steam engine drive, 482
stuffing boxes, 488, 499
suction pipe, 488
testing, 488
throttling, 484, 488
troubles, 488
turbine type, 483
vertical vs horizontal, 484
volute-type, 483
water-hammer damage, 781
wells, 248
Chalk, porosity, 78
volume-weight 678
Chambers, aqueduct, 300, 529
plug drain valves, 297
Channels, overflow, location 91
open, 269–279, (see *Open channels*)
proportions, 93
spillway, requisites, 146
Charleston, evaporation, 33
Goose Creek floods, 62
rainfall 5
reservoir, floating bottom 110
Chartrain dam, data, 164, 179
Chattanooga, rainfall, 5
Check valves, 451, 452
cross-connections, 403
industrial connections, 403
loss of head, 770
substitute, 443
Chemicals, 653–661
application, 666–676
concrete protected, 801
costs, 658
dry feed, 670
flow measurement, 792
formulas, 676, 677
materials attacked, 669, 801
specifications, 653
stream gaging, 42, 745
well tests, 74
Chenab river weir, 155
Cherokee dam, 146
Cheyenne, Wyo, rainfall, 5
Chezy formula, 754
open channels, 272
Chicago, intakes, ice, 83
tunnels, 292
tests, 767
pumping costs, 474
rainfall and temperature, 16
Chicopee standpipe, 558
Chili, rainfall, 21
Chimneys, 516–520
brick, 517
life, 516
capacity, 516
cost, 517
function, 516
height, 516
life, 836
Chimneys, lightning protection, 512
metal, life 335
Monoyer, 518
proportioning, 516
reinforced concrete, 518
steel, 516
temperatures, 519
China, rainfall, 23
Chloramine, disinfection, 662
Chloride, of lime, 662
water analysis, 597
Chlorinated lime, 662
Chlorination, tastes and odors, 663
Chlorinators, 674, 675
Chlorine, 662
algicide, 665
atomic weight, 678
coagulation aided, 654
decolorizer 654
formula 677
gas, pressures, 676
solubility 583, 663
liquid feeding, 673
map 586
package 677
pipe cleaning effect, 763, 813
in rain water, 601
volume, 678
water analysis, 600
significance 587
test, precision 738
weights 663, 678
Chloroform, formula, 676
Chlorophyceae, algicides, 679
growth 638
odor 614
Chromium, atomic weight, 678
Cincinnati, filters 743
high pressure fire system, 402
intake tunnels, 293
rainfall 4
water analysis, 615, 616, 620
Circles, area 752
circumference, 752
mathematical properties, 752
Circumference, circles, 752
Cisco dam, foundation, 145
Cisterns, rain water 695
City, population studies, 828
streets, pipe storage, 329
Clackamas River, dam, grouting, 127
Clark, hardness degrees, 597, 681
Clathrocystis, odor, 614
Clay, eroding velocity, 106, 276
ground water in 75
open channels, 270
porosity, 78
settlement in water, 106
size classification, 793
sluicing, 210
well sinking, 235
Cleaning, water mains, 421
chlorine effect, 763 813
Clearing, reservoirs, 108
Cleveland, filters, 744
high-pressure fire system, 402
intake tunnels, 293
subsiding basin, 642
water, tastes and odors, 612
Cloudbursts, extent, 93
Coagulants, 653–661 (see also *Chemicals*)
application, 666–676
costs, 658
formulas, 676
quantities, 657
Coagulating basins, 640–644
vs filters, 731
use, 659
velocities, 660
Coagulation, 650–661
adsorption, 650
alkalinity, theoretical, 654
chlorine aids, 654
colloids, theory, 651
colored waters, 654
contact baffles, 697
corrosion effect, 661

Coagulation, defined, 650
efficiency, 659
mixing, 659, 660
period, 660
pH-concentration, 651
optimum value, 661
precipitation, 651
purpose, 650
rapid filters, 660
slow filters, 660
soft waters vs hard, 652
storage effect, 633
vegetables retard, 652
water softening, 685
Coal (see also *Fuel*)
anthracite, 507, 513, 514
bituminous, evaporative power, 507
consumption, duty relation, 478
cost, 474
range, 831
evaporative power, 507
feed-water heaters save, 510
heat units required, 477
kinds, boiler effect, 507
mixed, 515
vs. oil, 512
pulverized, 515
pumping costs, 512
sampling, 514
specifications, 513
weight, 512
Coal-gas, tar, coating, 319
disinfectants, 631
pitch, coating, 315
Coatings, cast-iron pipe, 388
reinforced-concrete standpipe, 555
steel pipe, 315–322, 328 (see *Steel pipe*)
steel work, 814
wood-stave pipe, 371
Cobalt, atomic weight, 678
Cochituate, aqueduct, fouling, 296
river, run-off, 68
Coefficient, dilatation, 124
friction (see *Flow*)
storage, 90
uniformity, 80, 703
variation, stream-flow, 89
Coelosphaerium, odor, 614
Cold Spring dam, statistics, 218, 219, 220
Collection of water, 81–268
Colloids, 651, 652
Color in water, coagulation required, 650
pure, 583
reduction, chlorine, 655
rapid filters, 729
storage, 634, 635
standard desired, 637
tests, 585, 594
Colorado river, floods, 94
run-off, 69
silt, 62
Columbus, Ohio, filters, 710, 744
mixing tank, 660
water softening, 692
Columns, reservoir, 543, 548
Combustion, 506
Commercial pipe, 349
Companion flanges, 344
Compensation water, 93
Composite dams, 116
Compound meters, 464
Compression tanks, 502
Conconully dam, gates, 105
statistics, 220
Concord, rainfall, 3
Concrete, 793–803
Abrams' methods, 795
aggregate, 795
alkali soils, 373, 375, 797
aqueduct specifications, 283
blocks, masonry dams, 159
bonding new work, 159
cement, 793
loss, leaching, 122
core walls, 196
disintegration, 373, 375
durability, 797
Concrete, on earth foundation, 543
electrolysis, 812
erosion, 276, 797
forms, aqueducts, 288
dams, 159
freezing weather, 795, 796
fresh, withstands flood, 148
groined arches, 547
impermeability, 800
jackets, steel pipe, 322–325
leakage prevention, 555
limestone, aggregate, 813
lining, distribution reservoirs, 538
fouling, 297
steel pipe, 322–325
mixing by weight, 796
mixtures, conduits, 376
proportions, 795
paints, 800
paving, 200
pipes, 373
placing, rate, dams, 159
in winter, 795
proportioning, 795
rich, permeability, 800
sand, 793
setting, temperature, 122
slump test, 795, 796
specifications, 794
stresses, Amburson practice, 146
surfaces, tar coating, 798
temperature cracks, 122
tensile strength, 123
tile, 373
water, alkali, 797
waterproofing, 799 801
wells, 234
Condensers, 510
Conduits, 280–300, (see *Aqueducts*).
aquatic growths, 297
diversion, 85
earthquakes, 828
horse-shoe, flow, 769
hydraulic elements, 749, 752, 753
masonry, capacity, 767
flow, 767
monolithic concrete, 378
vs open channel, 269
pressure, 280
reinforced-concrete, 374–376
semicircular concrete, 379
steel pipe, (*see Steel pipe*).
subaqueous, 325, 416
twin, steel, 329
Conglomerate, ground water in, 76
Conifers, forestry, 111
Connecticut river, dam data, 161, 169
run-off, 44
Constant-angle arch dam, 137, 141
Construction, interest charges, 832
Continental Divide, rainfall and run-off, 60
Contraction joints, dams, 123
flumes, 278
Controllers, filters, 716, 731, 732
motor, 453, 524
Converse joint, pipe, 352
Conversion tables, 815
Coolgardie pipe, 318, 325
Copper, atomic weight, 678
coagulants affect, 669
corrosion, 808
poisoning, 587
service pipes, 432
sulfate, 665
algae dose, 297
formula, 677
package, 677
volume-weight, 678
Coquitlam dam, statistics, 218
Core, drill method, wells, 223, 229
drills, well sinking, 229
hydraulic fill dam, 211
loam, 195
masonry, 196
concrete, 196
grouting, 197
paving substitute, 198

Core, masonry, reinforced concrete, 198
position, 194
puddle, 195
purpose, 193
rock-fill dams, 189, 195
sheet piling, 194
Corfino dam, rapid construction, 135
Corporation cocks, 431
head lost, 434
Corrosion, boilers, 681, 687
bronzes, 807
cast-iron pipe, 387
chemicals, 669
Gerhard's studies, 348
metals, 808
pH value relation, 661
service pipes, 431
sodium silicate, 656, 661
soil, 429
steel pipe, 325
Costilla Creek dam, construction, 208
outlet, 94
Couplings, steel pipe, 352
wood pipe, 364
Covers, wells, 235
Cracks, flow of liquids, 771
Crenothrix, 588
Creosote, metal paint, 321
wood pipe, 369, 371
wood tanks, 569
Cresol, disinfectant, 632
Crest, gates, flat-slab dams, 146
movable, spillways, 103
roller, 104
Crib, intake, 81
Denver, 261
Crispin air valves, 447
Crops, rainfall intercepted, 66
water-table relation, 72
Cross, reducing, flanged, 392
standard flanged, 345
Cross River dam, concrete blocks, 159
data, 164
grouting foundations, 126
ice thrust, 118
temperature cracks, 123
uplift, 119, 164
water control, 158
Croton, aqueduct, tunnels, 292
dam, statistics, 218
flow, dry years, 65
rainfall, 4, 5
reservoirs, operation, 86
water analysis, 623
river, run-off, 47, 64, 67, 68, 70
water, treatment, 625
watershed, rainfall and temperature, 12
Croton Falls dam, data, 164, 179
ice thrust, 118
temperature cracks, 123
uplift, 119, 164
Crustaceae, growth, 639
Cryptomonas, odor, 614
Crystal Springs dam, leak, 122
Crystalite, zeolyte, 687
Crystalline rocks, ground water in, 77
Cubic, centimeters per L., to milligrammes, 584
feet per second, equivalent, 815
Culverts, aqueducts, 284
pipe, 407
rainfall data, 1
wood, creosoted, life, 371
Curbs, wells, 233, 631
Current meters, 464, 465
pipe measurements, 459
stream gaging, 40
Curtains, dams on porous foundations, 153
Curves, cast-iron pipe, 407–411
channels, hydraulics, 276
frequency, 6
steel pipe, 330
Cut-off walls, masonry dams, 126
Cyanophyceae, algicides, 679
growth, 639
odors, 613, 614
Cyclopean masonry, 802
economy, 159
Cyclotella, odors, 613, 614
Cylinder, capacities, 820
formula, arch dams, 136
hydraulic, gate valves, 444
oil, specifications, 480
pipe, 376
pumping engines, calculations, 480
thick hollow, formula, 827
Cylindrical gate valves, 453
Cypress, wood-stave pipe, 359, 360

D

Dale Dike dam, statistics, 220
Dams, adaptability, 116
Ambursen, 145
principle, 116
cellular, 141–146
composite, 116
curved masonry, 135–141, (see *Arched dams*).
earth, 193–221 (see *Earth dams*).
earthquake damage, 189, 828
failures, 117
flat-slab, 145
vs. multiple-arch, 142
principle, 116
foundations, pervious, 149–157
studies, 118
hollow, 141–146
economy, 117
porous foundation, 116
principle, 116
ice, damage, 117
thrust, 117
masonry, 116–188, (see *Masonry dams*).
multiple-arch, 142
gunite, 119
overflow, 146–148, 161, 162
aprons, 147, 152, 153
ogee face, 151
stilling pool, 274
surcharged, 145
water cushion, 148
on pervious foundations, 149–157
curtains, 153
design, 157
fore apron, 153
hollow, 116
hydraulic gradient, 151
rear apron, 152
repairs, 150
sheet piling, 153
talus, 155
types, 150
plan, curved vs. straight, 117
regulating, open channels, 276
rock-fill, 189–192
adaptability, 116
cores, 189, 195
solid gravity, (see under *Masonry dams*).
spillway, 146–148, 161, 162
state control, 160
steel, durability, 812
timber, 116
types, 116
Danube river, run-off and sediment, 63
Darcy, ground-water flow, 78
pipe flow, 754
Darley leak locator, 425
Davenport, Iowa, rainfall, 5
Davis Bridge dam, construction, 209
spillway, 94
statistics, 219
Deactivation, 431, 695, 810
Deaeration, corrosion, 387
Dean valve control, 443
Death rates, typhoid fever, 592
Debt, liquidation, equal payments, 835
Decalso, zeolyte, 687
Decomposition, nitrogenous, 602
Deer Flat reservoir, paving, 202
Deferrization, 699–701
filtration rate, 716
Memphis, 649
Defiance, O., water softening, 693
Degasification, 695
Dehri weir, 150, 156

de Lavaud, cast-iron pipe, 395
Delaware river, floods, 94
  water analysis, 617, 618, 619, 620
Delivery of water, Part III, 269–581
Demanganization, 701
Denver, Cheesman, storage reservoir, 91
  conduit, 375
  infiltration gallery, 260
  rainfall, 4, 5, 18
  temperature, 18
Depreciation, sinking-fund reserve, 835
  waterworks, 830
Depth, critical channels, 271
  mean, canals, 274
Derwent reservoir, seepage, 126
Des Moines, infiltration gallery, 261, 265
Detectaphone, 425
Detector, pipe, 426
Detroit, filters, 742
  high-pressure fire system, 402
  rainfall, 5, 16
  subsiding basin, 642
  temperature, 16
Diabase, porosity, 78
Diagram, logarithmic, 757
  mass, stream flow, 88, 92
Diamond drills, wells, 229
Diaphragm, flow measurements, 42
Diatomaceae, algicides, 679
  odor, 614
Dickens, run-off formulas, 65
Dictyosphaerium, odor, 614
Diesel engines, 527
  centrifugal pumps, 483
  operating costs, 521
Differential intake, 83
Dikes, (see *Earth dams*).
Dilatation, coefficient, 124
Dinobryon, odor, 614
Discharge, river, (see *Run-off*), 63
Disinfectants, application, 666–676
  heat, 661
  solution formula, 631
  sunlight, 661
Disinfection, 661–666
  excreta, 631
Disk meters, 463, 465
Distillation, 695
Distribution of water, (Part IV), 381–581
Distribution reservoirs, 533–570
  appurtenances, 536
  baffles, 536
  check valves, 536
  circular, merits, 542
  classes, 538
  columns, 543, 548
  construction plant, 541
  control valves, 536
  cost, 530
  depth of water, 538
  earth cover, 544
  earthquakes, 828
  effluent pipe, 536
  essentials, 524
  fire pressures improved, 533
  floors, 539, 543
  flow line, elevation, 633
  foundations, 543
  function, 533
  gate chambers, 536–538
  influent pipe, 536
  leakage, 541, 544
  life, 836
  lining, 538–541
    function, 534
  location, 533
  masonry roofs, 709
  open vs. covered, 538
  overflows, 536
  paving, 539
  pollution, 538
  protection, sanitary, 625
  vs. pumping, 522, 534
  reinforced-concrete, 542
    advantages, 534
    lining, 539
  in rock, 542
Distribution reservoirs, roofs, 544–547
    flat-slab vs. groined arch, 547
    floating, 544
    groined arches, 544
    masonry, 709
    vs. open, 538
    typical, 710
    wooden, 544
  shallow vs. deep, 538
  size required, 534
  skimming weirs, 536
  slab roof, 710
  slope paving, 539
  walls, 541, 544
    typical, 710
  water-level recorders, 569
Distribution systems, 398–435
  air valves, 398, 420, 446–448
  arterial system, 398
  bell-and-spigot joints, 412–416 (see *Cast-iron pipe*).
  blow-off valves, 420
  calking, 412
  cast-iron pipe (see *Cast-iron*).
    vs. steel, 399
  cement joints, 415
  cemeteries, 407
  check valves, 452
  cleaning, 421
    chlorine effect, 763, 813
  computations, 399, 773–780
  connections, hydrants, 457
    wet, 419
  corrosion studies, 809
  cost data, 405
  cross-connections, 403
    check valves, 453
  culverts, 407
  dependability, 402
  design, 399
  direct pumping, 469
  drinking fountains, 430
  earthquakes, 828
  electrolysis, 428–430
    cause, 428
    remedies, 429
  excavation, 404–406
  extensions, costs, 832
  failures, 402
  feeders, secondary, sizes, 398
  filling, 412, 777
  fire protection, 401–403
    costs, 402
    fire streams, 401
    industrial, 403
    insurance rates, 399
  fire service pressure, 400
  flow calculations, 399, 773–780
  freezing, 420
  gate valves, 436–446
  gridiron system, 399
  high-pressure, 393, 394, 396, 403
  house-to-house inspections, 426
  hydrants, 455–458
    spacing, 401
  industrial fire protection, 403
  joints, cement, 415
    factory-made, 394
    lead, 412, 413, 415
    lead removal, 413
    lead wool, 416
    lead-hydro-tite, 415
    leadite, 413
    making, 412–416
    metalium, 414
  leakage, 425–428
    control, 425
    detectors, 425
    high-pressure, 402
    records, 426–428
  losses in distribution, 423–428
  maintenance, 419–423
  map, 398, 440
  marshes, salt, 418
  materials, 399
    cast iron, 381
  meters, 459–468

Distribution systems, operation, 419–423
parallel lines, flow, 775, 776
pipes (see also *Cast-iron pipe*).
on bridges, 419
Chlorine effect, 763, 813
cleaning, 421
cutting, 412
jointing, 412–416
laying, 403–407
bends, 407–411
costs, 405
curves, 407–411
depth, 398
tests, 411
line raising, 420
locator, 425
sizes, 398
submerged, 416–419
pitometer surveys, 424, 426
pressures, 399
isolated section, 441
regulation, 448
pumping, 469, 470, 472
direct, 469
vs. gravity, 470
railroad crossings, 407
requirements, Bd. Fire Underwriters, 398
general, 398
services, 431–434 (see *Services*).
sewers, interference, 381
relations, 406
sizes, 398
sleeves, 420
sprinkler systems, 403
stream crossings, 416 419
submerged pipe lines, 416 419
surveys, 424, 426
electrolysis, 429
waste, 425
thawing, 421
trench dimensions, 404, 405
trenching machines, 406
valves, location, 440
sizes, 437
waste, 423
control, 426
surveys, 426
water unaccounted for, 423
watering station, 430
wet connections, 419
wood-stave pipe, 358, 367
Dive culverts, 284
Divers, manhole size, 82
Diversion, conduits, 85
dams, pervious foundations, 149 157
Divining rod, futility, 72
Dix River, dam, 191
reservoir, piers, 104
Dodge City, Kan , rainfall, 5
Dolomite, porosity, 78
Don Pedro dam, dense face, 119
Dorr, agitator, 660
clarifiers, 640
Douglas fir, wood-stave pipe, 360
Draft, forced, 506, 516
reservoir, defined, 87
studies (see *Yield*).
Drainage, area (see also *Watershed*).
defined, 87
earth dams, 205, 217
hydraulic fill dams, 217
wells, masonry dams, 119, 160
Dresser joints, 340
Drills, core, wells, 229
Drinking fountains, 430
Drop-down, open channels, 273
Droughts, 23
Druid Lake dam, statistics, 218, 220
Drum gates, on flat-slab dams, 146
Duluth, Minn., rainfall, 5, 17
temperature, 17
Dunnings dam, data, 161
Duocil, zeolyte, 687
Duplex pump, 474, 477
Duration curves, stream flow, 42
vs. mass curves, 89
Duty, safe station, 531
steam pumping plants, 478

E

Earth dams, 193–221
adaptability, 193
berm, 203
burrowing animals, 208
classification, 193
construction, 202–209
contents, 208
cores, 193–198
design, 202–209
dimensions, 218–221
drainage, 205
earth placing, 207
earthquakes, 828
failure, 208, 209
overtopping, 93
seepage, 205
foundations, 205
freeboard, 202
vs. gravity masonry, 116
highways on, 202
hydraulic consolidation, 208
hydraulic-fill, 209–218 (see *Hydraulic fill*).
imperviousness, 204
leakage, 207
life, 836
materials, 203
merits, 193
outlets, 205
paving, 198–202
requirements, 193
rolling embankment, 206, 207
saturation plane, 204
settlement, 203, 214
shrinkage, 203, 214
slopes, 203, 213
protection, 199
slides, 202, 203, 214
statistics, 218 221
stream control, 205
successful, 218–221
top dimensions, 202
volumes, 208
weight, 204
Earth, eroding velocity, 106
pressure, fluid equivalent, 541
masonry dams, 122
tests, 215
sluicing, 216
weight, 204
Earthquakes, 828
chimney design, 520
dam foundations, 125
damage to dams, 189, 828
rock-fill dams, 189
Earthwork, compacting, 207
steel pipe, 329
East Canyon Creek dam, composite type, 116
data, 139
described, 141
East Park dam, contraction joints, 123
Eastport, Me , rainfall, 5
Echo bridge, aqueduct, 290
Economizers, fuel, 510
Eel river dam, foundations, 125
overflow, 147
Efficiency, commercial, 471
filters, 737
pumps, 471
Ejector, sand, 712
El Paso, Texas, rainfall, 5
Elbow, flanged standard, 345, 391
loss of head, 769, 770
wood-stave pipe, 367
Eleanor dam, 144
Electric, drive, well pumps, 248
power, 522–525
thawing pipes, 421
Electrolysis, 428–430
cause, 429
concrete, 812
conditions, 429
effect, 428
insulation, thawing, 421
kinds, 428
leadite, 413
reinforced-concrete pipe, 375

Electrolysis, remedies, 429
  steel pipe, 301, 317, 326, 399, 811
  surveys, 429
  water meters, 465
  welded joints, 354
  wood-stave pipe, 360
Elephant Butte, dam, concreting rate, 159
  construction, 160
  contraction joints, 123
  Ensign valves, 453
  gate friction tests, 806
  grouting, 128
  outlet, 105
  sand cement, 794
  spillway, 94
  uplift, 119
  reservoir, silt, 107
Ellsworth dam, rapid construction, 146
Elm, ornamental planting, 111
Elwha dam, foundations, 125
Embankments (see also *Earth dams*)
  aqueduct, 285
  contents, 208
  grassing, 112, 114
  homogeneous, 193
  pipes through, 205
  rolling, 206, 207
  settlement, 203, 214
  shrinkage, 203
  slopes, 203
  weight, 204
Enamel, bitumastic, 317, 388
Engineering geology, 85
Engines, Diesel, 527
  centrifugal pumps, 483
  operating costs, 521
  gas, 526, 527
  centrifugal pumps, 483
  vs Diesels, 528
  fuel supply, 512
  operating costs, 521
  gasolene, 526
  internal-combustion, 520, 526 529
  pumping, 469–532, (see *Pumps* and *Pumping engines*)
  steam fire, delivery, 401
  operating cost, 521
Equalizing reservoir (see *Distribution reservoir*)
Erie, Penn , rainfall, 5
Erosion, safe velocities, 276, 753
Escanaba dam, data, 161, 170
Escondido dam, flood flows, 190
Esopus creek, rainfall and temperature, 10
  run-off, 68
Estacada dam, grouting 197
Estimates, precision, 747
Eternit, 374
Eudorina, odor, 614
Evaporation, 26–37
  boilers, 506
  data, value, 26
  factors, 26
  forest influence, 110
  laws, 26
  open channels, 269
  pan tests, 33
  records, 31 35
  residue, water analysis, 598
  soil, 35
  swamp, 85
Examination of water, 584 601
  acidity, 598
  alkalinity, significance, 586
  test, 598
  bacteriological 588
  media, standard, 600
  value, 584
  carbon dioxide, 598
  chemical, 585, 595
  form of results, 594
  value, 584
  chloride, 597
  chlorine, test, 600
  color, significance, 585
  test, 595
  dissolved oxygen, test, 599
  fungi, 588
Examination of water, hardness, 680
  significance, 586
  test, 596, 680
  units, conversion, 597, 681
  hydrogen-ion concentration, 587
  vs ice, 614, 622
  iron, 597
  manganese 597
  micro-organisms 587
  microscopical, 585
  molds, 588
  nitrates, significance, 586
  tests, 596
  nitrites, significance, 586
  tests, 596
  nitrogen, significance 585
  object important 584
  odor, significance, 585
  test, 595
  oxygen consumed, significance, 585
  test, 595
  *p*H value, 587
  physical 585, 595
  plankton, 588
  poisonous metals 587
  polluted, example, 591
  precision of results, 738
  residue on evaporation, 598
  samples, collecting 593
  standard methods 593 601
  turbidity significance, 585
  test, 594
  typical analyses, 620–623
  wholesome example 591
Excavating, hydraulic 216
Excavation, c i pipe, 404 406
  masonry dams 125
Excreta, disinfection 631
Expansion joint pipe, 393

F

Facing, (see *Paving*)
Factory, fire protection, 403
  intake 83
Fanning formula, run off, 65, 284, 285
  curve, 61, 65
Feed-water heating, 509
Feeds, chemical 667
Fences, Catskill Works, 105
Ferric sulfate, 655
Ferrous sulfate, coagulant, 655
  dry feed 670, 672
  formula 677
  package, 677
  turbidity removal 657, 659
  volume-weight, 678
Filter, 702
  accessories, 731
  contamination, protection, 625
  domestic 719
  double, 697
  drifting sand, 697
  effluents, water analysis, 618
  gages, 732, 733
  gravel, 703
  preparation, 705
  intermittent, 697
  loss of head gages, 732
  mechanical, 702
  medium, adsorption, 652
  operating tables 733
  operation, 733 738
  statistics, 735 736
  plants, (see *Filtration plants*)
  plates, brass, failure, 807
  preliminary, 695
  rain-water, 695
  rapid, (see below)
  rate controllers, 731
  sand, 703
  preparation, 705
  schmutzdecke, 702
  scrubbers, 695
  slow, (see below)
  supervision, 736
  tanks, concrete protected, 801

Filter, test day, 737
  water softening, 691
Filter, rapid, 716–731
  air binding, 734
  air wash, 729
  areas, 720
  arrangement, 720
  bacteria removal, 729
  barrel, 719
  bottoms, 724
  vs. coagulating basins, 731
  color removal 729
  defined, 702
  Gammage bottom, 725
  gravel, size, 723
    thickness, 723
  gravity, 716, 717, 720
  gutter elevation, 728
  Harrisburg, 723
  high-velocity wash, 726
  loading, 729
  loss of head, 726
  mud balls, 734
  operating costs, 738
  operating tables, 733
  pressure, 716–720
  principle, 716
  pumps, wash water, 729
  rate controllers, 731
  rates, 720, 733
  sand, 720
    expansion, 727, 728
    growth, 734
  sludge disposal, 729
  small, 719
  strainer system, 723
  turbidity removal, 729
  typical, data, 741–744
    drawings, 721, 722
  Venturi controllers, 732
  washing, 726–729
    principle, 716
  wash-water, quantity, 738
    tank, 728
    troughs, 733
  Wheeler bottom, 723, 725
Filters, slow, 705–716
  air binding, 716
  aluminum hydrate, 745
  bacterial film, 650
  beds, rectangular, sizes, 708
  Blaisdell machine, 714
  cleaning, 711–715
  cycle, 715
  defined, 702
  filtration rate, 716
  gravel, 709
  intermittent, 720
  loss of head, 715
  masonry roofs, 709
  Nichols separator, 714
  operation, 733
    cost, 738
  period, 715
  principle, 707
  raking, 716
  rate, 733
  regulation, 716
  sand, 709
    turbidity, 705
  scraping, 716
  structural features, 705
  typical plants, 739, 740
  underdrains, 708
Filtration, 702–745, (see also *Water treatment*)
  aeration relation, 649
  cost, 738
  efficiency, 737
  plant, clear-water basins, 533
    controllers, 453
    life, 836
    tank design, 541
    valve operation, 443
    walls, 541, 543
  rapid, 716–731
  rates, equivalents, 735
  sampling, 736
Filtration, slow, 705–716
    vs. rapid, 708
  statistics, 736
  vs. watershed patrol, 625
Financing, waterworks, 829–836
Fineness, modulus, 793
Fir, wood-stave pipe, 360, 369, 370
Fire, engines, delivery, 401
  insurance schedules, 399, 400
  line meters, 464
  protection, 401–403
    costs, 402
    fire streams, 401
    industrial, 403
    insurance rates, 399, 400
  pumps, piston speed, 480
  service, centrifugal pumps, 485
    pressures, 400
  stream, standard, 401, 402
    tank height, 535
  water consumption, 402, 572
Fish, copper sulfate kills, 666
  screens clogged, 82
Fishing, in reservoirs, rules, 626
Fittings, cast-iron pipe, 382
    flanged, 345, 389, 391
  centrifugal pumps, 487
  steel, superheat, 509
FitzGerald, evaporation formula, 27
  yield studies, 87
Fixtures, pipe sizes, 433
  water waste, 423
Flanged cast-iron pipe, 345, 389, 391
Flanges, gaskets, 441, 807
  screwed, 355
  standard, 390
  steel pipe, 340, 342–346
Flap valves, 453
Flashboards, 102
  automatic, 104
  diversion weirs, 156
  pins, stress, 155
  storage gained, 96
  typical installation, 155
Flexible joints, pipe, 394
Floats, stream gaging, 40
Flocculation, mixing tanks, 660
Flood, diagrams, 61, 95
  estimates, 65, 93, 95
  flows, 60
    rock-fill dams, 180
  hydrographs, 42, 100
  spreading, wells, 74
  source, 38
  vs. spillway capacity, 94
  stream gaging, 41
Flooding, wells, 74
Floor, covers, gate houses, 300
  distribution reservoirs, 539, 543
  groined-arch, 543
  reinforced-concrete reservoirs, 543
Flora, bacterial, determination, 589
Flow, Bernoulli's theorem, 751
  capillary tubes, 756
  channels, 271–276
  distribution system, 399
    calculations, 399, 773–780
  exponential formulas, 754
  flood, 60, 190
  line calculations, channels, 271–276
  liquids, cracks, 771
  masonry aqueducts, 297, 767
  measurement, 39, 792
  nozzle, 459
  in pipes, Bernoulli's theorem, 751
    formulas, 754–769
    hydraulic elements, 749, 752, 753
    relative discharge, 748
    temperature effect, 756
  stream, 38–70
  underground waters, 77
  wells, 238, 241
Flumes, 276–279
  metal, canal linings, 270
    coatings, 321
Fluorescein, ground-water studies, 72
Fluorine, atomic weight, 678

Flushing, pipes, 421
Foaming, boilers, 682
Folsom dam, data, 161, 170
Foot valves, 452
Forbes Hill reservoir, gate-chambers, 537
Forchheimer, ground-water flow, 78
Forest, evaporation factor, 27
transpiration, 37
water table, 73
Forestry, 110–112
Formaldehyde, disinfectant, 632
Forrest Park dam, statistics, 220
Fort Davis, Texas, rainfall and temperature, 19
Ft. Sill, Okla., rainfall, 5
Foundation, dams, studies, 118
embankments aqueduct, 285
grouting, 126
masonry dams, 125–128
pervious, dams on, 149–157
Fountain, aerators, 645
drinking, 430
water consumption, 572
Framingham (Sudbury River) Mass., rainfall, 3
Francis weir formula, 791
Frazil, 84
Freeboard, masonry dam, 128
Freezing, distribution system, 404
gate valves, 440
pipes, 404, 420
wood-stave pipe, 358
French pipe, 395
Frequency, curves, rainfall, 6
index, 608
Fresno, temperature and rainfall, 22
Friction, bronze on bronze, 455
hydraulic (see *Flow*).
Frost, depth, 404
Fuel, 512–515
coal (see *Coal*).
consumption, pumping engines, 478
cost, 512, 513
economizers, 510
oil vs. coal, 512
pulverized, 515
pumping cost, relation, 512
reserve required, 512
saving, superheat, 509
specifications, 513
weight, 512
Fuller, Weston, flood studies, 93
Fungi, 588
Furens dam, data, 164, 179
Furnaces, forced draft, 506

G

Gabbro, porosity, 78
Gages, filters, 732, 733
loss of head, 732
rain, 1
rivet, 303
Gagings, canal, 272, 275
stream, 39
records, 42
Galleries, infiltration, 260–265 (see *Infiltration*).
subterranean storage, 263, 264
Gallon, equivalents, 815, 818
imperial, equivalents, 815
Galvanizing, 813
service pipes, 432
Gas, engines, 526, 527
centrifugal pumps, 483
vs. Diesels, 528
fuel supply, 512
operating costs, 521
mains, distribution system, 406
producers, 527
vs. steam, 526
weight, conversion, 584
Gases, absorbed, water, 584
in ground waters, 603
in rain water, 601
solubility, 583
temperature, 584
weight, 584
Gaskets, 392
flanged, 441, 807
paper, 807
Gasoline, engines, 526
storage, 512
weight, 512
Gate, chambers (see also *Gate, houses*).
Forbes Hill reservoir, 537
roofs, truck loads, 827
crest, flat-slab dams, 146
houses, aqueduct, 529
floor covers, 300
heating, 105
micro-organisms, 297
in multiple-arch dam, 142
plug valves, 297
reservoir, 206
slow filters, 707
movable crest, dams, 104
shear, 455
sluice (see *Sluice gates*).
Gate valves, area of opening, 437
boxes, 440
care, 440
chattering, 441
closed, pressure, 441
controls, 443
cost range, 831
cylindrical, 453
disks, stresses, 440
double-disk vs. solid gate, 436
electrical operation, 443
filters, 732
reservoirs, 536
freezing, 440
gaskets, 441
hand operation, 442
high-pressure, dams, 454
hydraulic operation, 444
inclined stems, 105
indicators, 443
insertion by machine, 441
inspection, 404
location, 440
loss of head, 772
lubrication, 441
maintenance, 440
makers, 436
maps, 440
operation, 442–446
electrical, 443, 536, 732
water hammer, 782
ordering, 436
packing, stuffing boxes, 441
power to open, 442
remote control, 443
reservoir outlets, 454
seats, bronze, 805
erosion, 812
size, selecting, 437
solid-gate, makers, 436
vs. double-disk, 436
standard, 436
makers, 436
steel pipes, 330, 334
stem, 437
throttling, 436, 441
universal-pipe joint, 396
vaults, 440
Gatun dam, statistics, 218, 220
Gear, drive, 520
Diesel engines, 529
Gem Lake dam, 144
rebuilt, 142
Geography, run-off factor, 39
Geology, engineering, 85
run-off factor, 38
water-bearing, 75
Geophone, 425
Germany, infiltration systems, 264, 265
Germs, disinfection, 661
Gibraltar dam, arch action, 135
spillway, 94
Gilboa dam, construction, 160
masonry, 802
section, 174
stepped face, 148
stone facing, 160
Gileppe dam, data, 164, 180
uplift, 119, 164

Gilsonite, 315
Glacial deposits, southern border, 76
Glaciers, moraines, ground water, 72
Gleno dam, failure, 145
Glenodinium, odor, 614
Glenwild dam, statistics, 219
Gneiss, ground water in, 77
porosity, 78
Godaveri weir, 150
Gohna Lake, dam failure, 189
Goldbeck, pressure cells, 215
Goose Creek, floods, 62
dam, statistics, 218
Gooseneck, services, 431
Gorge, preglacial, wells, 77
Grade tunnels, gradients, 281
ground-water admitted, 291
use, 280
Grains per gallons, to p. p. m., 594, 678
Grand river, dam, gates, 104
flow velocity, 60
Granite, dam foundations, uplift, 120
ground water in, 77
porosity, 78
Granite Reef, weir, 151, 153, 154
Granite Springs, dam, data, 164, 180
Graphite, coating, steel, 317
lubricant, 479
paint vs. asphalt, 801
steel standpipes, 562
Graphitic corrosion, 387
Grassing, 112–114
dam slopes, 199
Gravel, discharge coefficient, 705
eroding velocity, 106, 276
filter, 703, 709, 723
flow of water, velocity, 79
packing wells, 238
rapid filters, 723
sizes, classification, 793
slow filters, 709
water-bearing, 75
Gravity dams, 116 (see *Masonry dams*).
Great Lakes, intakes, 83
pollution, 626
Great Northern Power Co. dam, 161, 170
Gridiron, distribution system, 399
Grit chambers, 640
Grizzlies, sluicing, 216
Groined arches, 544–547
roof, 709
Ground storage, 71
reservoirs, 86
Ground Water, 71–80
aeration, 647, 649
albuminoid ammonia, 585
appearance, 601
bacteria, 603–606
coli, 605
travel, 629
vitality, 637
capillary rise, 73
carbon dioxide, 603
removed, 649
color, 601
contamination, 628
durability of supply, 71
vs. filtered, 628
flow, 77
free ammonia, 585
gases, dissolved, 603
level, fluctuations, 73
mineral content, 606
nitrites, 586
occurrence, 71
odors, 602
organic matter, 602
origin, 601
purification, natural, 620
rainfall absorbed, 72
replenishment, 72
rocks, 73
storage, 639
supplies, investigating, 72
protection, 628
vs. surface waters, 71
tastes, 602
Ground Water, temperatures, 602
terminology, 71
utilization, 71
velocity measured, 74
yield, 74, 75
studies, cost, 71
Grout, 126, 803
mixer, 127
Grouting, core walls, 197
dam foundations, 126
pressure tunnels, 294
Grubbing, reservoirs, 108, 637
Grunsky, evaporation formula, 33
run off, 66, 95
Gun-crete, dam repairs, 122
Gunite, on dams, 119, 143
flumes, 278
slabs, paving, 112
Gypsum, porosity, 78

H

Haiwee dam, statistics, 218, 220
waves, 199
Hakodate, Japan, rainfall, 23
Hale, F. E., bact. coli index, 590
Hale's Bar dam, data, 162, 170
erosion, 148
Hammer welding, 348
Hardness of water, 680
carbonate, 680
non-carbonate, removal, 655
permanent, 680
significance, 586
temporary, 680
test, 596
precision, 738
total, 680
units, conversion, 597, 681
Hartford, filters, 740
rainfall, 5
storage reservoirs, 91
Havana, rainfall, 23
Hawaii, floods, 62
rainfall, 21
Hazen, coefficient of variation, 89
evaporation rule, 35
flow of ground water, 77, 79
yield studies, 87, 89
Headworks, arrangement, 280
location, 105
Health laws, watersheds, 625, 626
Hearn, run-off formulas, 65
Heat, disinfection, 661
Heating, feed-water, 509
Hell Gate dam, silt, 107
Hemet dam, data, 164, 180
Hemlock, forestry, 111
ornamental planting, 111
wood-stave pipe, 360
Henshaw dam, construction, 206
core, 197, 212
hydraulic fill, 210
materials, 211
outlet, 205
paving, 204
settlement, 215
statistics, 220
Hermastic, 317
Hetch-Hetchy, aqueduct, Red Mt. bar siphon, 324
dam, (see *O'Shaughnessy, Cal.*).
Hickory, forestry, 111
High-pressure fire systems, 394, 402
hydrants, 456
Highways, earth dams, 202
relocation, reservoirs, 104
Hijar dam, data, 165, 181
Hinckley dam, statistics, 220
uplift, 119
Hogsheads, Hurst's rule, 815
Holland, Mich., concrete wells, 234
ground-water yield, 74
Hollow dams, 141–146
Holyoke dam, data, 162, 171
ogee shape, 147
Honey Lake dam, statistics, 218, 219

Hope, Ark., rainfall, 5
Horse Creek dam, construction, 207
Horse troughs, water consumption, 572
Horsepower, boiler, 506
brake, 471
theoretical, 471, 473
water, equivalents, 819
Horseshoe dam, data, 144
Hot-air engines, 502
Hotel, water consumption, 571
Housatonic river, run-off, 48
Houses, pump, wells, 249
spring, 268
Huacal dam, design, 141
Hub flanges, 342
Huffman dam, core, 213
Humidity, evaporation effect, 26
Hydrants, 455–458
Board of Fire Underwriters, 456
care, 440
connections, 457
cost range, 831
designs, 456
freezing, 440, 457
life, 836
loss of head, 400
maintenance, 456
ordering, 456
pressures, 400
rentals, 402
spacing, 401, 457
subsurface, 458
supplied from two directions, 533
universal-pipe joint, 396
Hydrated lime, 656
Hydraulic, computations, 747–792
fill dams, (see below).
jump, 274
chemicals, mix, 659
spillway channels, 94
pipe (see under *Pipe*).
weight, 305
radius, 749, 752
channels, 269, 274
ram, 502–505
valves, 453
Hydraulic-fill dams, 209 218
advantages, 210
characteristics, 209
construction, methods, 209, 215
safe, 210
core, 211
costs, 210
desiderata, 210
disadvantages, 210
drainage, 217
economy, 210
fines, 212
foundations, 214
materials, 210
pumping, 216
rock fill, 213
settlement, 214
sheer boards, 216
slopes, 213
sluicing, 216
Hydraulics, 747–792
open channels, 271–276
wells, 238
Hydrogen, atomic weight, 678
-ion concentration, 650, 651
corrosion, 810
significance, 587
sulfide, aeration, 646
reservoirs, 636
solubility, 583
weight, 584
weight, 584
Hydrographs, 42, 100
Hydrolene, 315
Hydrology, impounding reservoirs, 85

I

Ice, damage, dams, 117
evaporation, 34, Fig. 6.
expansive force, 421
Ice, intake troubles, 84, 105
physical properties, 118
stream gaging, 41
thrust, dams, 117
existing dams, 118, 163–167
multiple-arch dams, 143
water analysis, 614, 622
Impellers, centrifugal pumps, 485
Impounding reservoirs, 85–93, (See *Reservoirs, storage*).
Increaser, standard flanged, 345
Incrustants, water analysis, 587
Index, frequency, 608
India, flood estimates, 93
rainfall, 23
watersheds, run-off, 65
weirs on pervious foundation, 149–157
Indianapolis, Ind., rainfall, 5
Indicators, pH value, 652
Industrial, fire protection, 403
Inertol, concrete paint, 801
Infiltration galleries, 260–265
clogging and cleaning, 260
examples, 261–265
German methods, 264
Los Angeles, 293
pipes, collectings, 260
pollution, 606
rates of flow, 260
tunnels, 293
vs. wells, 260
yield, 260
Ingersoll-Rand formula, air lift, 252
Ingot iron, corrosion, 534, 811
pipes, 338
Insurance, schedules, 399
Intakes, 81 84
crib, Denver, 260
Great Lakes, 82
ice troubles, 84, 105
infiltration gallery, 260-265
large, 81
pipe, 82, 418
bellmouth, 81
port areas, 82
requirements, 81
reservoirs, 82, 105
river, 82
sanitary protection, 625
screens, 81, 82
submerged pipe, 418
surges, 82
tunnels, 82, 292, 293
capacity, 767
Internal-combustion engines, 520, 526–529
Iodine, 665
atomic weight, 678
Ionization, 650
Iron, atomic weight, 678
bacteria, 588
corrosion, 387
cast, (see *Cast iron*)
colloidal, precipitated, 652
corrosion, 809
ingot, corrosion, 534, 811
pipes, 338
sulfate solutions, 655
in water, analysis, 597
color, 636
test, precision, 738
wrought, corrosion, 811
Irrawaddy river, run-off, 63
sediment, 63
Isohyetal map, 4
Italy, water consumption, 571
Ithaca, dam, data, 139
rainfall, 5
subsiding basin, 642

Jacksonville, high-pressure fire system, 402
James river, run-off, 52, 53, 54, 69
Jersey City, conduit, test data, 763
storage reservoirs, 91
Jet, condensers, 510
Jetting method, wells, 223, 228

Johnson hydraulic valve, 453
O'Shaughnessy dam, 105
Joints, bell-and-spigot, (see *Cast-iron pipe*).
contraction, flumes, 278
masonry dams, 123
pipes, 377
expansion, (see *contraction* above).
pipe, butt, 302
commercial, 352–355
contraction, 377
Dresser, 340
flanged, 389
flexible, 394, 416
high-pressure, 355
reservoir linings, 539, 540
riveted, 301–310
design, 306–310
shrunk and peened, 340, 342
sleeve, 340
slip, 340
spiral-riveted, 339
Jump, hydraulic, 274, 659
Junction dam, construction, 210
pumping, 216
statistics, 221

Kansas City, subsiding basin, 642
Keechelus dam, core, 197
road roller, 207
sheeting, 194
Kensico dam, concreting rate, 159
construction, 160
contraction joints, 123
data, 165
ice thrust, 118
masonry, 802
pressure diagram, 129
stability investigation, 129
thermophones, 123
uplift, 119, 165
Kensico reservoir, effluent aqueduct, 379
forestry, 112
silting, 106
stumps, 109
Keokuk dam, erosion, 148, 162, 171
foundations, 125
Kilowatt, equivalents, 817
-hours per month, 472
Kistna weir, 150
Klangenfust, rainfall, 4
Kuichling, run-off, 95, 284, 285
curve, 61
studies, 65
Kutter's formula, 271
canals in earth, 271

L

L-shaped walls, 542
Labor, price range, 831
Ladders, standpipes, 536
La Grange, dam, (see *Turlock*).
reservoir, silt, 107
Laguna weir, 150, 156, 157
Lahontan dam, construction, 207
material, 204
spillway, 94
statistics, 218, 219
Lake Cheesman dam, data, 139, 165
profile, 180
Lake, circulation periods, 635
deposits, ground water in, 77
sanitary patrol, 626
thermometry, 634
water analysis, 623
waters, bacteria, 611
filtration rate, 716
index of frequency, 608, 609
wind currents, 635
Lake Ontario, bacteria, 612
Lake Spaulding dam, thermophones, 123
La Loutre dam, forms for concrete, 159
gate house, 105
ice thrust, 118
uplift, 119

Lamé's formula, thick cylinders, 827
Lap, joints, flanged, 340
riveted joints, 301
Larch, forestry, 111
ornamental planting, 111
Las Vegas dam, flashboards, 103
Lavaud, de, cast-iron pipe, 395
Lea V-notch meter, 459
Lead, atomic weight, 678
corrosion, 808
hydro-tite, 415
joints, cast-iron pipe, 412, 413, 415
vs. cement, 415
holding power, 411
vs. leadite, 414
vs. metalium, 414
solid, 405
pipes, corrosion, 387, 636
poisoning, 587, 809
red lead, 562
removal, 413
service pipes, 432
wool, 416
Leadite, 413
electrical resistance, 429
electric thawing, 421
Leakage, detectors, 425
distribution system, 425–428
control, 425
detectors, 425
records, 426–428
reinforced-concrete standpipes, 549, 550, 554
reservoir, 108
steel pipe, 356, 428
Levees, grassing, 112
Lewiston, Idaho, rainfall, 5
Liege, infiltration galleries, 260
Light, water absorbs, 584
Lime, chemical properties, 655
chlorinated, disinfectant, 632
corrosion, 669
treatment, 387
dry feed, 671, 672
excess, bactericide, 685
feeding device, 671–673
and ferrous sulfate, 655
hydrated, 656
dry feed, 670
formula, 677
package, 677
waterproofing, 560
meters affected, 465
-soda water softening, 682, 685, 693
volume-weight, 678
Limestone, dam foundations, uplift, 120
ground water in, 76
porosity, 78
reservoir location, 108
Limnology, 634
Linden, English, ornamental planting, 111
Lining, distribution reservoirs, 538–541
function, 534
open channels, 269, 270
Linville dam, statistics, 219
Liters, equivalents, 818
Lithium, atomic weight, 678
Little Falls, filters, 743
Little Fork river, run-off, 69
Little Horse Creek, dam, statistics, 220
Loam, cores, 195
eroding velocities, 276
Loch Raven, dam, underdrains, 126
uplift, 119
reservoir, silting, 107
Lock Joint pipe, 376–378
Lock-bar pipe, 311
vs. cast-iron, 381
fabrication, 312
failure, 312
laying, 326
merits, 312
specifications, 311
Log booms, waves, 201
Logarithmic, diagrams, 757
tables, 837
Long Island, ground-water yield, 74
Los Angeles, rainfall, 20, 22

Los Angeles, storage reservoirs, 91
temperature, 20, 22
Loss of head, canal sections, 274
at entrance, 274
pipes, formulas, 754–769
Venturi meter, 461
Louisville, filters, 742
rainfall, 5
Lowcock, evaporation formula, 33
Lowell, Mass., rainfall, 3
Lower Otay, dam, failure, 189, 191
reservoir, silt, 107
Lubricants, pumping engines, 479
Lubrication, gate valve, 441
Lueger, ground-water flow, 78, 79
Luitweiler pumps, 502
Lynchburg, wood-stave pipe, 361
Lynn, Mass., storage reservoirs, 91
Lysimeter experiments, 35

M

McCall's Ferry dam, data, 162, 172
McCloud river, run-off, 69
McCracken machines, pipes, 373
McMath, run-off, 95
curve, 61
Machine, cleaning, pipes, 421
dry feed, 670
pipe-bending, 352
tapping, 420
trenching, 406
valve insertion, 441
Madaya weir, 150, 156
Madison, Wis., rainfall, 5
Magnesium, atomic weight, 678
salts, formulas, 676
solubility, 659
Mains, 398 435 (see *Distribution system*)
filling, 412, 777
Mallomonas, odor, 611
Manchester, Mass., standpipe, 550, 553, 554
Manchester, N. H., rainfall, 3
Manganese, atomic weight, 678
bronze, 806
water analysis, 597
test, precision, 738
Manhan river, run-off, 68
Manholes, size, divers, 82
steel pipes, 333
valve, 440
Mannesmann, seamless tubes, 355
Manning, formula, 754
Map, distribution system, 398, 440
isohyetal, 4
Maple, forestry, 111
Norway, ornamental planting, 111
ornamental planting, 111
sugar, ornamental planting, 111
Marble, porosity, 78
Marine structures, deterioration, 375
Marklissa dam, uplift, 119
Marquette, Mich., rainfall, 5
Marshes, c. i. pipe, 388
salt, pipe in, 418
Masonry, 793-803
aqueducts, 280-300, (see *Aqueducts*).
core walls, 196
cyclopean, 802
dams, (see *Masonry dams*).
economy, 159
friction coefficient, 128
laying, dams, 158
temperature cracks, 122, 123
weight, 131, 802
Masonry dams, 116–188
arched, 135–141 (see *Arched dams*).
construction, 157
defects, 148
cut-off walls, 126
use, 153
design, 128–135, 136
drainage wells, 119, 160
earthquakes, 828
facing, concrete blocks, 159
failures, causes, 146
flow over, 790
Masonry dams, formulas, 128
freeboard, 128
gravity, solid, (see *solid gravity* below).
height raised, 160
hollow, 141-146
ice thrust, 117
life, 836
masonry laying, 158
notches, flow, 791
openings for stream, 157
on pervious foundations, 149-157
pressure lines, 128
seepage reduced, 126
shearing stresses, 133
solid gravity, 118–135
architectural treatment, 118
contraction joints, 123
curved in plan, 116
design, 128 135
methods, 147
disintegration allowance, 121
vs. earth, 116
earth, backing, 119
foundations, 126
pressures, 122
economy, lack of, 117
excavations, 125
failures, cause, 128
foundation pressure, 128, 131
foundations, 125 128
geology studies, 118
grouting foundations, 126
ice thrust, 128
leaks, 122
non-overflow, design, 128 135
top width, 133
overflow, 146 148, 161, 162
pressure distribution, 134
public favors, 116
requisites, 118
vs. rock-fill, 190
stability equations, 132
stream control, 157
temperature, allowance, 121
cracks, 122
effects, 122
thermophones, 122
thickness, ineffective, 118
toe, earth fill, 122
types, 116
typical, 161 187
upward pressure, 118, 119, 128, 161–167
watertightness, 122
wind pressure, 119
zones, design, 133
types, 116
vacuum effects, 147
water control, 157
weight, 802
Mass diagram, spillway, 100
stream flow, 88, 92
Massachusetts, towns, water consumption, 582
Materials life, evaluation, 836
price range, 831
Matheson pipe, 352, 353
Mathis dam, 146
Matrimony vine, embankments, 113
Median, 63
Meer Alum, dam, 141
Melosira, odors, 613
Memphis wells, 240, 255, 649
Merala weir, 150, 156
Merced dam, statistics, 220
Mercuric chloride, disinfectant, 632
Mercury, atomic weight, 678
pressure, equivalents, 818
Meridion, odor, 614
Merrimac, river, sediment, 63
standpipe, 550, 552
Merriman, evaporation, 27
Metals, 805-814
corrosion, 808
Metalium, 414
Metcalf and Eddy run-off, 95
Meter, boxes, 467
current, stream gaging, 40
service (see below).

Metering, benefits, 459, 467
statistics, 576–581
waste control, 426, 467
water consumption affected, 468, 573
water unaccounted for, 423
Meters, service, 459–468
accuracy, 465
advantages, 459
boxes, 467
compound, 464, 465
current, 464, 465
defective, replacement, 465
dimensions, 465
disk, 463, 465
fire-line, 464
head loss, 463
lengths, 465
life, 836
makers, 468
operating costs, 467
oversized, 465
proportional, 464
qualifications, 463
rates, 468, 834
replacement, 465
rotary, 464
setting, 467
Simplex, 463
sizes, 466
slippage, 466
specifications, 465
testing, 466
torrent, 464
turbine, 464, 465
types, 459
underregistration, accounting, 423
velocity, 464
Venturi, 459–463
Meyer, evaporation formula, 30
Miami, Fla., high-pressure fire system, 402
Miami Conservancy District, construction plant, 202
cores of dams, 211, 212, 213
dams, seepage, 194
sluicing, 217
statistics, 219–221
excavation 126
flood flows, 60
grassing, 113
hydraulic fill dams, 210, 221
pressure tests, dams, 215
rainfall rates, 24
spillway design, 93
Microorganisms, algicide, 679
storage effect, 638
surface waters, 608
water, 587
Microns, equivalents, 587
Microscopy, 587
Middleboro, deferrization, 700, 701
filters, 710
tank, 556
Milan, rainfall, 4
Miles City, Mont., rainfall, 5
Miller-Brownlie, formula, wells, 240
Milligrammes, per L to c c, 584
Million, gal daily, equivalents, 815
Milton dam, core, 197
paving, 200
statistics, 219
Minatare dam, paving, 200
Minerals, suspended in water, 611
Miner's inch, definition, 816
Minneapolis filters, 742
Minnesota river, run-off, 69
Mississippi river, floods, 94
ice troubles, 84
run-off, 63, 69
sediment, 63
water, sedimentation, 643
Mixing basins, 659, 660
Mobile, Ala., rainfall, 5
Molds, water microscopy, 588
Molybdenum, atomic weight, 678
Mondidier, France, rainfall, 23
Monel metal, 807
Monoyer chimneys, 518
Morena dam, flood flows, 190
vs. Roosevelt dam, 190
Morgantown, W Va., rainfall, 5
Morris dam, statistics, 218
Mortar, linings, steel pipes, 322–325
sand, 793
Motors (electric), a.c., 522
controllers, 524
d.c., 522
life, 836
slip-ring induction, 523
speeds, common, 522
squirrel-cage induction, 522, 524
starters, 524
supersynchronous, 524
synchronous, 523
valve operation, 443
Mountain Dell dam, 142, 144
Movable crest, spillways, 103
Mud, balls, rapid filters, 734
well sinking, 235
weight, 131
Multiple-arch, dams, 142 145
Munich, ground-water yield, 74
Murphy, run-off curve, 61
Muscoot dam, data, 162, 172
flashboards, 103
Myers' formula, run off, 284, 285

N

Narrora weir, 149, 150, 151, 154, 156
National Board of Fire Underwriters, centrifugal pumps, 483
duplex pump rules, 474
electric power, 522
factory yard system, 403
fire flow, 401
fuel reserve, 512
hydrants, 456
spacing, 401
requirements, pipe system, 398
sprinkler systems, 403
Standard Schedule for Grading Cities, 399, 401
storage, size, 534
turbo-centrifugals, 498
National Matheson Joint Pipe, 353
Necaxa dam, failure, 210
statistics, 218, 219
Needle valves, 453
high velocities, 105
O'Shaughnessy dam, 105
New Bedford, Mass., rainfall, 3, 11
temperature, 11
New Croton, aqueduct, fouling, 297
leakage, 296
tunnels, 292
dam, data, 165
grouting foundations, 126
ice thrust, 118
temperature cracks, 123
New England, chlorine map, 586
rainfall data, 2
N. E. W. W. A., cast-iron pipe specifications, 381, 389
hydrant specifications, 455
meter specifications, 465
service standards, 432
thawing, report, 421
water consumption, forms, 572
report, 423
New Haven, Conn., storage reservoirs, 91
New Orleans, cold periods, 825
filters, 741
hydrant regulations, 456
rainfall, 4, 5, 18
temperature, 18
water analysis, 620
New York City, cold periods, 825
high-pressure fire system, 402
storage reservoirs, 91
valve standards, 436
New York State, chlorine map, 586
Newark, N. J., conduits, flow tests, 764
rainfall, 5
storage reservoirs, 91

Newark, O. mixing tank, 660
Niagara river, flow, 60
Nickel, atomic weight, 678
Nile river, floods, 94
run-off, 63
sediment, 63
Nitrates, water, significance, 586
tests, 596
Nitrification, 602
Nitrites, water, significance, 586
tests, 596
Nitrogen, as albuminoid ammonia, significance, 585
tests, 595
atomic weight, 678
cycle, 602, 603
free ammonia, 585, 595
test, 595
solubility, 583
water analysis, significance, 585
weight, 584
Notches, flow through, 791
Nozzle, aeration, 647
fire, discharge, 401, 402
flow, 459
hydraulic sluicing, 216
Numbers, powers, 752
Nuts, dimensions, 341
weight, 344

O

Oak, forestry, 111
ornamental planting, 111
pin, ornamental planting, 111
Oakland, East Bay Water Co., storage reservoirs, 91
high-pressure fire system, 402
Obsidian, porosity, 78
Ochoco dam, sluicing, 216
statistics, 220
Odors, aeration removes, 645
chlorination, 663
degrees, described, 595
iron bacteria, 588
organisms, 613
reservoir stripping, 637
water, tests, 585, 595
Oester dam, uplift, 120
Ogden, Utah, rainfall, 5
Ohio river, bacteria, 616
floods, 94
run-off, 69
sediment, 63
turbidity, 615
Oil, crude, weight, 512
cylinder, specifications, 480
engines, 527 (see also *Diesel*).
fuel supply, 512
fuel, vs coal, 512
cost, 513
range, 831
Diesel engines, 528
duty relation, 478
vs. gasolene, 526
requirements, 513
tests, 529
hydraulic cylinder, 445
linings, reservoirs, 200
lubricating, 479
Oiling, canal linings, 270
Okhla weir, 150, 151, 152, 153, 156
Oklahoma City, filters, 741
floods, 93
Olive Bridge dam, data, 163, 176
grouting, 127
Omaha, rainfall, 5
Oolite, porosity, 78
Open channels, 269–279
Chesy formula, 272
vs. conduit, 269
contractions, flow, 771
critical depth, 271, 274
curvature effect, 276
economy, 269
evaporation and seepage, 269
flow, calculations, 271–276
measurement, 792
Open channels, flumes, 276–279
gagings, 272, 275
hydraulic jump, 274
hydraulics, 271–276
Kutter's formula, 271
lining, 269, 270
mean depth, 274
non-uniform flow, 273
regulating dams, 276
in rock, 269, 272
section, 269
semicircular conduit, 379
surging, 275
U. S. Reclamation Service, 269, 270
velocities, safe, 276
velocity absorbed, 271
Organisms, (see also *Microorganisms*).
odors imparted, 613
in pipes, 297
reservoir stripping, 637
stored waters, 633
Orifices, coagulants mixed, 659
compensating, 708
discharge, 771
plate, 459
strainer, rapid filters, 724, 725
tanks, chemical feed, 668
Oscillaria, in ice, 613
O' Shaughnessy (Cal.) **dam,** concreting rate, 159
construction, 160
excavation, 126, 160
Johnson valves, 453
outlets, 105
section, 175
O' Shaughnessy, Ohio, dam, ice thrust, 118
section, 175
Ottertail river, run-off, 69
Outlet, earth dam, 205
reservoir, 82, 105
Overflow, channels, location, 94
proportions, 93
dams, 146–148, 161, 162
weirs, 93–104
aqueducts, 281
Oxyacetylene, welding, 347
Oxygen, absorption, sprays, 646
atomic weight, 678
consumed, (water) significance, 585
precision, 738
test, 595
dissolved corrosion, 809
test, 599
solubility, 583, 599
weight, 584
Ozone, water treatment, 665
basis, 601

P

Packing, gate valves, 441
turbines, 499
Paddy Creek dam, statistics, 218
Padua, rainfall, 4
Paint, concrete, 800
deterioration, 319
tar-cement, 321
Painting, standpipes, 561, 562
steel work, 814
Pandorina, odor, 614
Parkersburg, infiltration gallery, 260
Parts per million, 594
to grains per gallon, 678
Pathfinder dam, data, 139, 165, 181, 187
gate friction tests, 806
reinforced-concrete core, 198
Patrol, sanitary, 625
Paving, distribution reservoirs, 539
earth dams 198–202
concrete, 200
core-wall substitute, 198
costs, 202
gunite slabs, 112
reinforced-concrete, 201
stone, 200
waste channel, failure, 94
Payments, equal, for debt and interest, 835
Pebbles, eroding velocity, 106
settlement in water, 106

Pedlar River dam, data, 162, 171
Peking, China, rainfall, 23
Penstocks, (see also *Steel pipe*).
corrosion, 325
screens, 299
Peoria, Ill., rainfall, 5
Pequannock watershed, rainfall and temperature, 13
Percolation, factor, diversion weirs, 149, 156
wells, 231
Peridinium, odor, 614
Period, coagulation, 660
Periyar dam, 166, 182
waste weir, 148
Permutit, water softening, 686
Petroleum, weight, 512
pH concentration, 651
acid and alkali solutions, 653
corrosion relation, 810
Phenol, coefficient, 631
disinfectant, 632
equivalent, 632
Philadelphia, cold periods, 825
ground-water yield, 74
high pressure fire system, 402
rainfall, 5, 13
temperature, 13
water analyses, 617, 618, 619, 620
Phoenix, Ariz., rainfall and temperature, 19
Phosphorus, atomic weight, 678
Pickling, steel pipe, 314
Pierson Act, New Jersey bonds, 832
Pilarcitos dam, statistics, 218, 219
Piling, sheet, core wall, 194
dams on porous foundations, 153
Pine, Austrian, ornamental planting, 111
Norway, ornamental planting, 111
red, forestry, 111
Scotch, forestry, 111
ornamental planting, 111
trees, forestry economics, 112
white, forestry, 111
ornamental planting, 111
wood-stave pipe, 360
Pins, flashboard, 103
Pipe, age, flow effect, 756
air-lift wells, 251
Armco, 338
bell-and-spigot, 389, (see also *Cast-iron pipe*).
jointing materials, 412–416
bending machines, 352
bends, combined, 408–411
loss of head, 769
Bonna, 376
branching, problem, 774
on bridges, 419
leadite, 413
protection, 404
steel, 334, 339
wood-stave, 372
Briggs standard, 349
capacity, (see *Flow*).
cast-iron, (see *Cast-iron pipe*).
cement, 373
cement-lined, 374, 388
capacity, 297
electrolysis, 429
centrifugal, 377
for chemicals, 670
circular, geometric properties, 747
hydraulic elements, 749, 752, 753
cleaning, 421
chlorine effect, 763, 813
concrete, 373
contents, 752, 820, 824
corrugated-iron, flow, 766
couplings, 352
culverts, 407
cutting, 412
drive, wells, 224
driving, wells, 231, 232
Eternit, 374
filling time, 777
fittings, flanged, 345
flow, (see also *Flow*).
formulas, 754–769
measurement, 792
Pipe, flow, relative discharge, 748
freezing, 420
wood-stave, 358
French, 395
hi-tensile, 383
hydraulic elements, 749, 752, 753
infiltration galleries, 260
ingot iron, 338
intake, 82
bell-mouth, 81
lead, corrosion, 387, 636
line vs. masonry aqueduct, 283
vs. pressure tunnels, 280
location, 426
Lock Joint, 376–378
long, 754
loss of head, formulas, 754–769
through masonry dam, 157
moss, distribution systems, 422
o. d., 349
dimensions and weights, 351
organisms, 297
parallel, equivalents, 775
plate metal, 301–357, (see *Steel pipes.*)
plumbing, sizes, 433
pressure, 376
refill stresses, 383
reinforced-concrete, 373–380, (see *Reinforced-concrete*).
riveted, 301–357, see (*Steel pipes*).
wrought-iron, 334–338
sand-spun, 395
screw, electrolysis, 352
service, 431–434, (see *Service*).
sewer, 373
flow, 768
well casing, 234
short 754
siphons, 779
sleeves, 420
small, fungi clog, 588
specials, welding, 346
spiral-riveted, 338–346
steel, 301–357, (see *Steel pipes*).
steel-cylinder, 376
stream crossings, 416–419
surges, 781
system computations, 773–780
threaded, 393
twin, flow problem, 776
vitrified, 373
discharge, 768
well casing, 234
volumes, 752, 820, 824
water hammer, 781–785
welded, 348–355, (see *Steel pipe*).
window, 217
wood-stave, 358–372, (see *Wood stave*).
wrought, 349
wrought-iron corrosion, 810
dimensions and weights, 350
riveted, 334–338
vs. steel, 349
Piston, pump merits, 476
speed, 480
Pitching, dams, (see *Paving*).
Pitometer, hydrant, 456
surveys, 423, 424
Pitot tube, 459
Pitt river, run-off, 69
Pittsburgh, filter, 739
rainfall and temperature, 15
water analysis, 620
Piute dam, sluices, 217
Planks, stop, 299
Plankton, water analysis, 588
Planting, ornamental, 111
Plants, aquatic, water consumed, 35
Plate, metal pipes, 301–357, (see *Steel pipes*).
steel, pipe, 310
standpipes, 560
Platinum, atomic weight, 678
Plattsburg dam, 145, 146
Plugs, cast-iron pipe, 404, 411
drain valves, 297
steel pipes, 333
Plumbing, pipe sizes, 433

Plumbism, 587, 809, (see also *Lead*).
Plunger, pump, 474
merits, 476
wells, 247
Pneumatic, systems, pumping, 502
Po river, run-off, 63
sediment, 63
Pohle system, air lift, 250, 251
Poisonous metals, water, 587
Pole-tool, drilling, wells, 225
Poplar, ornamental planting, 111
Population, studies, 828
Porosity, rock, values, 78
Portland, Ore., rainfall, 5
Potassium, atomic weight, 678
salts, formulas, 676
Potomac river, sediment, 63
run-off, 58, 63
Power, calculations, 471, 476
electric, 522–525
equivalents, 816
kinds, 520
pumps, 471
connections, 520
rates, 525
well pumps, 247
Powers of numbers, 752
Precast concrete pipe, 376–378
Pre-chlorination, 654
Precipitation, 1–37, (see *Rainfall*).
Pressure, aqueduct, gradients, 281
barometer heights, 472
conduits, 280
conversion factors, 818
distribution systems, 399
waste relation, 400, 426
equivalent, 817, 818
filters, 716–720
pipe, 376
regulating values, 448–451
Anderson, 451
choice, 449
location, 449
loss of head, 772
Ross, 449
types, 448, 450
use, 448
-regulators, 448–451
pumps, electric control, 525
relief valves, 450
screens, 82
steam, rule, 480
strainers, 82
tanks, 502
trapezoidal law, 134
tunnels, 291–296
Catskill aqueduct, 294, 295
depth of cover, 291
gradients, 281
leakage, 294
lining, 291
vs. pipe lines, 280
use, 280
water, upward, dams, 119, 161–167
Priest dam, core wall, 198
paving, 202
Prime movers, operating costs, 521
pumps, 474–482, 487, 520–529
Priming, boilers, 682
centrifugal pumps, 489
Private water companies, charges, 402
Probability, rainfall laws, 6
Proportional meters, 464
Protozoa, algicides, 679
growth, 639
odor, 614
Providence, mixing tank, 660
rainfall, 3, 5, 11
temperature, 11
Public watering station, 430
Puddle, 803
core, 195
Pulleys, belt drive, 520
Pulverized-fuel, 515
Pumping, altitude, 472
costs, 473, 832
pipe cleaning affects, 422
Pumping, direct system, 469
distribution reservoir, relation, 469
economics, 470
engines, (see below).
fuel cost relation, 512
vs. gravity supply, 470
head, 471
horsepower, 471, 473
hydraulic fill dam, 216
low-lift, cost, 738
machinery, makers, 470
types, 470
peak loads, 471
pressure, 472
vs. reservoirs, 534
stations, chimneys, 516–520
dry wells, 484
duty, safe, 531
economic design, 469
fire hazard, 402
hot well, 509
instruments, 512
suction wells, 530
superstructures, 529
suction, 472
wells, effect, 71
rates, 243
wood-stave pipe leaks, 359
Pumping engine,
condensing, water required, 511
crank-and-flywheel, 474
cylinder calculations, 480
defined, 471
duty, 478
erection, 486
fire department, 401
fuel consumption, 478
gasoline, 526
hot-air, 502
hydraulic ram, 502–505
life, 836
lubricants, 479
non-pulsating, 501
piston speed, 480
pneumatic systems, 502
power, 520–529
calculations, 471, 476
reciprocating, (see below).
Reeco, 502
rotary, 482
tests, 471
windmills, 505
Pumping engine, reciprocating, air chamber, 474
best, elements, 477
boiler horsepower, 506
vs. centrifugal, 476, 484
chemicals applied, 666
condensing, performance, 477
costs, 479
cross-compound, 475
cylinders, proportions, 480
discharge, theoretical, 480
duplex, 474, 477
efficiency, 472
mechanical, 477
electric drive, 520, 522, 523
flexibility, 471
high-duty, makers, 478
merits, 478
makers, 474
merits, 476
motors, 523
noise, 481
oil engines, 527
pipe connections, 481
power calculations, 476
proposals, 478
quadruple expansion, 475
reservoir head, 469
single, 474
slip, 481
steam pressure, 480
superheat, 508
triple-expansion flywheel, 475
triplex, 474
triplex compound, 475
vs. turbo-centrifugals, 490

Pumping engines, reciprocating, types, 474
uniflow, 475
water hammer, 476
Pumps, 469–532
air, boilers, 510
air-lift, 249–259 (see *Air-lift*).
aqueduct unwatering, 297
belt drive, 520
booster, 470
capacity, fires, 469
centrifugal, 482–497 (see *Centrifugal pumps*).
chain drive, 522
for chemicals, 670
mix, 659
cylinder calculations, 480
efficiency, commercial, 471
electric drive, 522–525
control, 469
fire-fighting requirements, 469
gear drive, 520
intake pipe, 82
Luitweiler, 502
manufacturers, 470
piston, merits, 476
pit, fluctuating rivers, 82
plunger, 474
merits, 476
wells, 247
pneumatic systems, 502
power, 471
calculations, 471, 476
prime movers, 474–482, 487, 520–529
rotary, 482
electric drive, 522, 524
selection, 471
service required, 469
single, 474
size of units, 471
turbine centrifugal, 248, 483
types, 470
wash-water, 729
well, 247–259
Punching, well sinking, 232

Q

Quadruple expansion, pumping engine, 475
Quarter bend, standard flanged, 345
Quartzite, defined, 76
ground water in, 76
porosity, 78
sand, specific gravity, 641
Quebec, Que. rainfall and temperature, 8
Quemahoning dam, core pond, 212
pumping, 216
settlement, 214, 215
statistics, 219
Quicklime, disinfectant, 632
dry feed, 670
formula, 677
Quicksand, well sinking, 236

R

Radium, atomic weight, 678
Radius, hydraulic, 747, 752
channels, 269, 274
mean, canals, 274
Railroad, crossings, distribution system, 407
relocations, reservoirs, 104
tanks, 568
water station, intake, 81
Rain, gages, 1
location, 2
storm, area affected, 93
water, characteristics, 601, 622
gases in, 601
supplies, 695
Rainfall, 1–25
altitude effect, 4
Continental Divide, 60
data, 2–23
precision, 2
estimating, 3
evaporation, 30
factor, 26
frequency curves, 6
Rainfall, ground absorbs, 72
ground-water yield, 74
laws, probability, 6
maxima rates, 24
maximum and minimum, 5
mean annual dependable, 6
mountains, 5
ocean effect, 4
rates, 815
records, American, 6–22
combining, 3
comparable, 6
dry years, 3
foreign, 21, 23
long term, 4
published, 6, 42
reliability, 4
unreliable, 2
variation, 64
annual, 5
run-off, factor, 38
tables, 43 60
soil evaporation, 36
variation, U S 5, 7, 23
Ram, hydraulic, 502–505
water shock, 781
Rate, controllers, 716, 731, 732
water supply, 468, 832, 834
Recorders, stream gaging, 41
water-level, 569
Red Bluff, rainfall and temperature, 22
Reducers, flanged, standard, 345, 391, 392
Redwood, wood-stave pipe, 360, 369, 370
Reforestation, 110
Regulating dams, open channels, 276
Reinforced-concrete, chimneys, 518
conduits, 374–376
flumes, 278
lining, reservoirs, 59
pipe, (see below).
standpipes, 548 556, (see *Standpipes*).
tanks on towers, 556
Reinforced-concrete pipes, 373–380
advantages, 374
carrying capacity, 374, 375
contraction joints, 377
cost, relative, 358
disadvantages, 374
durability, 375
electrolysis, 375
essentials, 373
failure, 374
fouling, 375
friction-head table, 755
leakage, 378
Lock Joint pipe, 376–378
mixtures, 376
monolithic, 375, 378
Precast pipe, 375, 376–378
rusting, 375
siphon, ribbed, 380
steel-cylinder pipe, 376
Reinforcement, bond, 551
corrosion, 324
electrolysis, 812
quantity, 542
rusting, 375
shrinkage, 543
temperature, 540
twisted vs. deformed, 542
Rennes, infiltration galleries, 260
Reserve storage, 533–570, (see *Distribution reservoirs*).
size required, 534
Reservoirs, automobile camps, 625
bathing regulations, 626, 627
boating, regulations, 626
capacity, evaporation effect, 35
circulation, temperature effect, 635
clearing, 108
covered, advantages, 639
cylindrical, capacity, 820
deep, oxygen depleted 636
distribution 533–570, (see *Distribution reservoirs*).
fences, 105
fishing, regulations, 626

Reservoirs, float controls, 451
grubbing, 108, 637
hydrogen sulfide, 636
impounding, (see *Reservoirs, storage*).
organic matter, source 637
outlets, 82
pollution, 625
rectangular, capacities, 822
retarding value, 99
soil stripping, 637
storage, (see *Reservoirs, storage*).
stripped, color effect, 637
stripping, 108
stumps removed, 637
sunlight effect, 661
temperature of water, 643
thermocline, 638
transition zone, 638
water analysis, 623
waters, filtration rate, 716
Reservoirs, storage, 85–93
bridge piers, 104
capacity calculations, 86
cemeteries, 104
compensation water, 93
costs, 87
floating bottom, 110
flood effect, 94, 99
flow line, economic, 85
foundation studies, 118
geology, 85
grassing, 112
ground storage, 86
headworks, 105
hydrology, 85
index of frequency, 609
intakes, 105
land costs, 87
large vs. several small, 86
largest in U S, 87
leakage, 108, 126
life, 836
minor structures, 104
outlet works, 105, 205
overflow prevented, 92
quality effect, 633–639
reforestation, 110
regulating effect on flow, 94
rise, and waste weir, 94
run-off and overflow, 98
sewers, 104
silting, 106
statistics, 91
storage, ratio, 634
units, 86
topography, 85
watershed development, 85–115
wind currents, 635
yield, 87–93
Rhine river, stream-flow, 65
Rhone river, run-off, 63
sediment, 63
Richmond, Va, rainfall, 5
Rio Grande river, run-off, 63
sediment, 63
Riprap, diversion weirs, 155
earth dam, 200
Riser pipes, tanks, 563
River, channels, Kutter's "n," 272
discharge, (see *Stream-flow*).
foreign, water analysis, 620, 621
intake, 82
mean depth, 274
run-off, (see *Run-off*).
sediment, 62, 63
sleeve, 352
stages, records, 42
supplies, protection, 625
velocity, high, 60
water, analysis, 620, 622
filtration rates, 716
Rivet, calking, 303
cold driven, 302
countersunk, cost, 303
edge distance, 303
gages, 303, 306
heads, flattening, 303
Rivet, holes, 302
pitch, 301, 303, 306
steel pipe, 302
for wrought-iron pipe, 336
Riveted joints, 301–310
Riveted pipes, 301–357, (see *Steel pipes*).
Riveting, field, pipe, 327
steel pipe, 302, 303
wrought-iron pipe, 336
Rivularia, odor, 614
Road rollers, pressures, 206, 207
Roanoke river, run-off, 55, 56, 69
Rochester, conduit, flow tests, 764
evaporation records, 32
Rock, channels, flow, 272
-fill dams, 189–192
adaptability, 116
cores, 189, 195
ground water, 73, 76, 77
open channels, 270
porosity values, 78
sluicing, 216
springs in, 266
Roller, crest, 104
Roofing, paper, steel coating, 318
w i vs steel, 335
Roofs, reservoir, 544–547
masonry, 709
vs open, 538
tanks on towers, 538
Roosevelt dam, curved plan, 125
data, 166, 182
vs Morena, 190
Root river, run-off, 69
Ross pressure regulator, 449
Rotary, meters, 464
method, wells, 223–227
pumps, 482
electric drive, 522, 524
Rotifera, growth, 639
Run-off, 38–70
calculations, 64
Continental Divide, 60
culvert areas, 285
defined, 38
diagrams, 61, 95
distribution curves, 42
elevation effect, 88
equivalents, 815, 816
estimates, 69
factors, 38
formulas, 61, 65, 285
rainfall, data for estimates, 66
tables, 43–60
records, 42
rainfall, calendar year, 2
variations, 64
relation to rainfall, data, 1
swamps, 87
units, 39
Rust, blister, 111
Ryves, run-off formulas, 65

S

Sacramento, filters, 725, 744
rainfall, 5, 22
river, run-off, 69
temperature, 22
Saddles, steel pipes, 333
St. Croix river, run-off, 69
St. Louis, cold periods, 825
cylinder oil specifications, 480
filters, 721, 741
rainfall, 5
subsiding basin, 642
water analysis, 620
St. Paul, rainfall, 4, 5, 17
storage reservoirs, 91
temperature, 17
Salmon creek dam, data, 139
Salt, marshes, pipe, 418
water (see *Sea, water*).
Salton sea, evaporation, 31
Salts, flow measurement, 792
formulas, 676
ionization, 650
solubilities, 664

Sand, analysis, 704
  cement, 794
  for concrete, 793
  effective size, 703
  ejector, 712
  eroding velocity, 106, 276
  filter, 703
    rapid, 720
    slow filters, 709
  fineness modulus, 793
  foundation, masonry dams, 140-157
  hydraulic-fill dam, 211
  open channels, 270
  percolation factor, 150
  porosity, 78
  rapid filters, 720
  separators, 714
  settlement in water, 106
  sizes, classification, 793
    determined, 703
    effective, 703
    separation, 705
  slow filters, 709
  spun pipe, 395
  subsiding velocity, 640
  turbidity, 704
  uniformity coefficient, 80, 703
  unsatisfactory, 794
  voids, determined, 705
  washing, 709
  water-bearing, 75
Sandstone, dam foundations, 120
  ground water in, 76
  occurrence, 76
  porosity, 78
San Andreas dam, statistics, 218
San Antonio, flood, 93
San Diego, storage reservoirs, 91
  rainfall, 5, 23
San Dieguito, dam, 144
San Francisco, earthquake, 828
  high-pressure fire system, 102
  rainfall, 4, 20, 22
  storage reservoirs, 91
  temperature, 20, 22
San Leandro dam, statistics, 218, 219
San Mateo dam, curved plan, 135
  data, 166
  earthquakes, 125
  uplift, 119, 166
Santa Barbara, infiltration gallery, 263
Santa Fé, rainfall, 5
Santa Maria dam, statistics, 218
Saratoga wells, fluctuation, 73
Sarco, 315
Scendesmus, odors, 613
Schaefer dam, materials, 211
Schist, ground water in, 77
  porosity, 78
Schizomycetes, algicides, 679
Schoharie reservoir, clearing, 110
Scioto, dam, data, 162, 172
  river, sediment, 63
Scituate reservoir, flashboards, 103
Scobey, flow, wooden stave pipe, 766
Screening, well, rotary method, 228
Screens, aqueduct, 297
  cleaning devices, 82
  intakes, 81, 82
  lifter, 298
  pressure, 82
  submerged, flow, 771
  well, 237
    damage, 244
Screw pipe, electrolysis, 352
Scrubbers, filters, 695
Sea water, cast-iron pipe affected, 416
  evaporation, 26
  fire use, 402
  freezing, 420
  oxygen dissolved, 599
  pressure, conversion, 818
  water analysis, 623
  wells, 245
Seattle, rainfall and temperature, 21
  storage reservoirs, 91
Second-feet, equivalents, 815
Sector gates, 104
Sediment, rivers, 62, 63
  settling rate, 106
  transportation, 106
Sedimentation, 633-639, 640-644
  basins, distribution reservoirs used, 53[illegible]
  theory, 640, 643
Seepage, evaporation includes, 26
  open channels, 269
Semibituminous, defined, 513
  sizes and analysis, 514
Semi-Diesel engines, 527
Semihydraulic, construction, dams, 209
Service pipes, 431-434
  brass, 432
    failure, 807
  cement-lined, 374, 432
  cleaning, 422
  connections, 431
    relative ease, 381
    tapping machines, 420
    wood pipe, 366, 367
  copper, 432
  corrosion, 431, 587
  friction-head, 760
  fungi clog, 588
  galvanized, 432
  grounding circuits, 432
  laying, 432
  lead, 432
  leakage, 428
  life, 431, 836
  materials, 431
  sizes, 431
  standards, 432
  thawing, 421
  welded, 348
  w i, specifications, 349
Service reservoirs, (see *Distribution reservoirs*)
Services, extensions, cost, 832
  illegal, 423
  water waste, 423, 424
Settlement, materials in water, 106
Settling basins, 640 644
Sevier bridge dam, core, 194
  failure, 205
  statistics, 220
Sewage, bacterial life, 610
  detection, water, 585
  disinfection, 631
  effluent, analysis, 591
  streptococci, 590
  on watersheds, 627
Sewers, distribution system, 406
  drop-down curves, 273
  flushing, water consumption, 572
  impounding reservoirs, 104
  pipe, flow, 768
Shafts, centrifugal pumps, 485
  spillway, reservoirs, 94
  tunnel, 291
Shale, dams on, 125
  ground water in, 76
  porosity, 78
Shear gates, 455
Sheet piling, cores, dams, 194
Sheeting, core wall, 194
Shenandoah river, run-off, 57
Sherburne Lakes dam, drainage, 205
  gates, 104
  statistics, 220
  stream control, 206
Sherman Island dam, uplift, 120
Shoes, wood-stave pipe, 361
Shoshone dam, 139, 166, 182
Shrubs, ornamental planting, 111
Shrunk and peened, joint, pipe, 340, 342
Shutter, (see *Flashboard*).
  stop plank, 299
Silicon, atomic weight, 678
Silt, allowance, Indian reservoirs, 105
  effect on algae, 297
  intakes exclude, 81
  percolation, 71
  pressure on dam, 122
  removable, subsidence, 640
  river records, 62, 63

[illegible] velocity, 640
[illegible]estry, 110
[illegible]s, 106–108
[illegible] atomic weight, 678
[illegible]plex, meters, 463
[illegible] prepared-joint pipe, 396
[illegible]nking fund reserves, 835
[illegible]phon, culverts, 284
  defined, 280
  gradients, 281
  hydraulic losses, 276
  pipe 779
    curves, 410
  reservoir outlet 205
  spillways 101
  steel pipe 307
Size effective 703
Slate, porosity 78
Sleeve, joints, 340
  pipe 420
  river, 352
Slichter, ground water flow, 78
Slip, joint, 340
  pumping engines 481
Slopes, earth dams 203 213
Slow filters, (see *Filters*)
Sluice gates, 454
  discharge, 746, 772
  for high heads 105
  seats bronze 805
  Stoney 104
Sluicing, 216
  clay 210
Smith system, infiltration 260
Smokestack, (see *Chimney*)
Snake River dam, 193
  statistics 219 221
Snow, evaporation 34 (Fig 6)
  measurement 1
  rainfall equivalent 6
  removal water consumption 572
  run off 38
  water analysis 622
Soap, hardness effect 681
Soda ash, 656
    dry feed 672
    formula 677
    package 677
    water softening 683
  caustic formula 677
    package 677
    volume weight 678
    water softening 684
Sodium, atomic weight 678
  carbonate soda ash volume weight 678
  chloride coagulant 655
    taste 602
  hydrate, volume weight 678
  hypochlorite 662
  salts formulas 676
  silicate corrosion 656 661
  thiosulfate volume weight 678
Sodom dam data 160 183
Softening, water 680 694 (see *Water softening*)
Soil, aeration 628
  alluvial springs 267
  bacteria 603
  corrosion 387 429
  eroding velocities 276
  evaporation 35
  organic matter 602
  permeability, 77
  porosity 78
  stripping 108
    reservoirs 637
Solids, suspended American rivers 63
  water carries 62
Solubilities, salts 664
Solution tanks, 669
Solutions, application 666 676
  rate of flow, table 669
Somerset dam, construction, 209
  materials, 211
  statistics, 218, 219
Sonofone, 425
Sonograph, 425
Sonoscope, 425
Sooke River conduit, 375 378
Soundings, stream gaging, 40
Sources, water supply, protection, 625–632
South Haiwee dam, cut-off, 195
Spanish River dam, water control 158
Specials, pipe steel 381
  welding, 346
Speed, specific 489
Spier Falls dam, data, 162, 166, 173, 183
Spillway, 93 104
  aqueduct 281
  backwash, 202
  capacity 93 94
  channel requisites 146
    velocity 276
  dam, 146 148 161 162
  discharge diagrams 99
  economic length 97
  gates blocking 191
  movable crests 103
  proportions 93
  rainfall data 1
  siphon 101
  surcharge at Oklahoma City 145
Spiral-riveted pipes, 338 340
Spokane, rainfall 5
Spray-nozzle aerators, 646 648
Spring Valley Water Co, cement joints, 415
  pipe coating 318
  storage reservoirs 91
  welded joints 352
  wrought iron pipe 335
Springfield, (Mass) filters 708 710 733
  steel pipe coating 316
Springfield, O softening plant 687
Springs, 266 268
  classification 266
  development 266
  geological conditions 267
  houses 268
  occurrence 71
  pollution 606
  in rock 266
  in soil 267
  water analysis 591, 622
    bacteria 604
    bact coli 605
    protection 630
  wells interfere 223
  yield 266
Sprinkler systems, 403
Spruce, Douglas ornamental planting 111
  forestry 111
  Norway ornamental planting 111
  white ornamental planting 111
  wood stave pipes 360
Spudding, wells 224
Stacks, 516 520 (see *Chimney*)
Standley Lake dam, statistics 219
Standpipes, earthquakes, 528
  lack of economy, 534
  ladders 536
  life 836
  pumping relation 469
  reinforced concrete 548 556
    architectural treatment, 556
    calculations 549
    cement gun 551
    coatings 555
    concrete proportions, 550
    construction 551
    dimensions, 549
    disfiguration 554
    floor-and wall joint 552
    leakage 549 550 554
    membranes 555
    merits 542
    reinforcing steel, 550
    repairs 555 556
    spalling 554
    vs steel 548
    stresses 549
    valve chamber 551
    waterproofing 550 554
  size required 535
  steel 558 562
    anchorage 558

Standpipes, steel, architecture, 561
dimensions, 558, 559
failure, 558
foundations, 558
large, 558
life, 561
makers, 558
painting, 561, 562
plates, 560
protection, 813
vs reinforced concrete, 548
shell, 560
specifications, 562
use, 534
surges, 783
use, 534
water hammer effect, 782
Starters, motor, 524
State, control of dams, 160
regulation, water supply, 829
Station, public watering, 430
pumping, (see *Pumping*)
Stauwerke gates, 104
Steam, 507
boilers, 506–512 (see *Boilers*).
consumption, turbines, 500
engine, operating cost, 521
vs. gas, 526
lbs per i h p., 472
pressure, rule, 480
pumps, (see *Pumping engines, reciprocating*)
turbines, 497–501
vs Diesel engines, 528
economy, 499
makers, 497
merits, 499
operating costs, 521
regulations, 498
stuffing boxes, 499
types, 498
use, 497
weight, 507
Stearns, yield studies, 87
Steel, cast, superheat, 499
coagulants affect, 669
corrosion, 809
-cylinder pipe, 376
flumes, 277
pipes, (see below)
standpipes, 558–562 (see *Standpipes*).
work, painting, 814
wrought, 349
vs. wrought iron, 335
Steel pipes, 301–357
air valves, 446, 447
anchorages, 330, 334
bends, 330
bridge, 334, 339
bump joints, flow effect, 765
capacity, 322, 324, 761–765
required, 280
vs cast-iron, 301, 399
cleaning, 314
closure pieces, 329
coating, 315-322
Angus Smith tar, 315
asphaltum, 318, 322
bitumastic enamel, 317
bituminous, centrifugal, 315
demerit, 325
blisters, 322
burlap coating, 318
comparisons, 315
Coolgardie pipe, 318
defects, 321, 322
electrolysis protection, 317
experiences, 326
failure, 325
field joints, 321
function, 315
graphite, 317
mineral rubber, 339
protection, 328
repairs, 321
roofing paper, 318
shop, 315
Spring Valley Water Co., 318
Steel pipes, coating, Springfield pipe, 3[illegible]
tar-cement paint, 321
tests, 316
U. S. Reclamation Service, 319
water-gas and coal-gas tar paint, 319
collapse, 310, 312, 357
commercial, 349–355
bends, 352
designations, 349
dimensions, 350
distinguished from wrought-iron, 349
installations, 355
joints, 352–355
nomenclature, 349
service pipe, 349
specifications, 349
weight, 350
concrete jacket, 322
concrete-lined, corrosion, 306
design, 313
conduit, capacity required, 280
connections, 333
corrosion, 325, 810
costs, 304
relative, 369
cross-sectioner, 357
curves, 330
cutting, 346
deformation, 356
details, 333
deterioration, 325
distribution system, 399
double-riveted, head and weight, 305
durability, 325
earth load, 356
earthwork, 329
electrolysis, 301, 326, 399, 811
preventives, 430
erection, 326–332
experience, 356
failure, 310, 312
air valves, 416
field, calking, 328
riveting, 327
flanges, 340, 342–346
flow, 322, 324, 761-765
gate valves, 330, 334
head lost, 433
high-pressure systems, 301
intake, 82
joints, riveted, 301–310, (see *Steel pipe, riveted*)
laying, 326–332
leakage, 356, 428
life, 326, 836
lock-bar, 311
capacity, 762
fabrication, 312
failure, 312
merits, 312
specifications, 311
manholes, 333
Mannesmann, 355
merits, 301
mortar lining, 322–325
oiling, 314
pickling, 314
pittings, 326
mill scale, 348
plates, 310
plugs, 333
repairs, 356
riveted, calking, 303
field, 328
capacity, 312, 758, 762–765
age effect, 764
vs. c. i., 301, 399, 762
table, 755
diameter, 311
vs c. i., 762
discharge, 762–765
distribution system, 399
flow, 761–765
friction head, 758, 761, 762, 765
joints, 301–310
butt, 301, 302, 309

Steel pipes, riveted, joints, calking, 303
circular seams, 301
costs, 304
design, 306–310
double riveting, 301
edge distances, 303
efficiency, 303
failure, 301
junction of seams, 309
lap, 301, 307, 308, 765
long. seams, 301
pitch of rivet, 303
rivets, 302, 303
stove-pipe, 304
lap, 301, 307, 308, 765
laying, 326–332
riveting, field, 327
rivets, countersunk, cost, 303
saddles, 333
seamless (see *welded* below)
shop calking, and testing, 306
riveting, 303
siphons, 313
gradients, 281
sizes, 311
specials, welding, 346
tapered, 304
tuberculation, 315 322, 765
spiral-riveted, 338 346
advantages, 339
coating 339
experience 339
flanges, 340
flow, 766
joints, 339
makers, 338
manufacturing, 339
patents, 339
specials, 339
stream crossing, 416–419
stresses, 338
thickness and weight, 341
swamps, 329
temperature effects, 356
testing, in field, 332
shop, 306
thickness, 310
tuberculation, 315, 322, 765
twin pipe lines, 329
valve connection, 330
weight, 301, 305
welded, 348 355
advantages, 348
bends, 352
disadvantages, 348
discharge, 765
failure, 347
installations, 355
joints, field, 352 355
manufacture 348
Matheson, 352, 353
o d, 349
dimensions and weights, 351
vs riveted steel, 348
specifications, 349
strength, 347
welding, 346, 347
well casing, 351
well casing, 234, 351
whitewash, 314
vs wood stave, 359
vs wrought-iron, 335, 349
Stokers, 508
Stone, paving, earth dam, 200
Stoney sluice gates, 104
Stony brook, run-off, 68
Stony River dam, 145, 146
failure, 125
flashboards, 103
Stop, disks, reservoirs intakes, 82
planks, 299
Storage, available, 86
bacteria, affected, 637, 638
reduction, 633
coefficients, 90
color reduced, 634, 635
deferrisation, 700
Storage, filtered waters, 639
gallery, 263, 264
ground, 74
reservoirs, 86
waters, 639
microorganisms, 638
natural, run-off, 39
period, 633
reserve, 533–570 (see *Distribution reservoirs*)
types, 534
reservoirs, 85–93 (see *Reservoirs, storage*)
systems, statistics 91
turbidity reduction, 633
units, 86
water 85 93, 533 570 (see *Reservoirs*)
quality effect, 633 639
Storm rates, 21
Stovepipe, casing 227
corrosion 246
joints, pipe, 304
method, well sinking 223, 225
Strainer, pressure 82
system, rapid filters, 723
well, 237
Strawberry dam, core, 198
settlement, 190
Stream, coefficient of variation, 89
control earth dams, 203
earth dams 205
masonry dams 157
crossings pipe 416 419
deposits ground water in 77
flow, 38 70 (see also *Run-off*)
calculations 64
data utility 39
defined 38
equivalents, 815
mass diagram 88
maximum, diagram, 61, 95
minimum, 64
records 41 63
10 year, fallacy, 65
seepage augments, 72
self purification, 607
gagings 39
records, 42
scour capacity, 62
underflow, 71
velocity high, 60
Street flushing, water consumption, 572
Stripping reservoirs, 108
Strontium, atomic weight, 678
Stuffing boxes, 488 499
valves 111
Stumps, removal 109
reservoir, 637
Submerged pipe lines, 416 419
Subsidence, in basins, 640 644
in storage reservoirs, 633 639
theory, 640, 643
Suction, feet, to vacuum 817
lift pumping, 472
wells pumping 520
Sudbury, aqueduct, bridges, 290
cleaning, 297
fouling 296
tunnel cave-in 290
dam, statistics, 219
reservoir, color reduction, 634
paving, 201
river, run-off 45 67, 68
watershed, rainfall, 4, 5, 9
temperature 9
yield, 87
Sulfate of alumina (see *Alum*)
Sulfur, atomic weight, 678
Sulfuric acid, package, 677
volume-weight, 678
Sunlight bleaching action, 635
disinfection, 661
Sunol aqueduct bridge, 290
Sunshine, data, 27
Superheat, benefits, 508
Superheaters, 508
Superheating, 508
Superstructures, pumping station, 529

Surface, condensers, 511
vs ground waters, 71
run-off, 38–70
vs wells, 222
Surges, open channels, 275
pipes, 781
Surveys, electrolysis, 429
water waste, 424–426
Susquehanna River, run-off, 50, 51
West Branch, run-off, 49
Sutro weir, 788, 791
Suviana, multiple-arch dam, 142
Swamps, c i pipe, 388
drainage, 636
evaporation, 85
grubbing, 109
run-off, 87
steel pipes, 329
yield effect, 85, 92
Sweetwater, dam, data, 139, 166, 183
height raised, 160
reservoir, silt, 107
Sylvester's process, 800
Synedra, odors, 613
Synura, odor, 614
tastes, 663

T

Tabeaud dam, statistics, 218, 220
Tabellaria, odor, 614
Talbot formula, run off, 284, 285
curve, 61
Tallulah dam, flashboards, 104
Talus, dam on pervious foundation 155
Tampa, Fla , rainfall, 5
Tank flanges, 346
Tanks, alum solutions, 669
cylindrical, calculations, 542
capacities, 820
merits, 542
filter, design, 541
mixing, filters, 660
rectangular, capacities 822
on towers, bottom, form, 563
dual tank, 534
earthquakes, 828
economy, 534
fire stream capacity, 535
industrial use 536
ladders, 536
piers, 565
reinforced concrete, 556
riser, frostproofing, 534
roofs, 538
size required, 535
sizes, 562
steel, 562 565
foundations, 564
pipes, 563
specifications, 563
standard, 564
towers, 563
vs wooden, 566
use, 534
wooden, 566–569
capacities, 567
creosoting, 569
life, 569
vs steel, 566
towers, 569
usefulness, 566
weight, 567
Tapping machines, 420
Tar, cement paint, 321
coating, c i pipe, 388
concrete surfaces, 798
steel pipes, 315, 319, 320
Tastes in water, chlorination, 663
Tatnuck Brook, run-off, 68
Tee, loss of head, 769, 770
reducing, flanged, 392
standard flanged, 345
Temescal dam, statistics, 219
Temperature, effects, masonry dams, 122
steel pipes, 356
evaporation, 29
Temperature, reinforcement, concrete, 546, 542
run-off factor, 38
sedimentation effect, 643
tropics, 29
U S records, 6–22
Tenax, solution, steel pipe, 317
Terrace reservoir dam, cost, 210
gates, 105
materials, 211
statistics, 219, 220
Texaco, 315
Thawing, pipes, 421
Thermometers, electric, in masonry, 123
Thermophones, masonry, 122
water studies, 634
Thirlmere dam, data, 166, 183
Threaded pipe, 393
Throttle dam, core, 196
overtopped, 202
statistics, 220
Throttling, centrifugal pumps, 484, 488
valves, 436, 441
removable, 82
Tidone Barrage, multiple-arch, 144
Tieton, canal, surges, 275
dam, core, 197
materials, 211
Tiles, concrete, 373
well casing 234
Till, defined, 76
ground water in, 76
Tillotson brook, run-off, 68
Timber, life, 369
Timbering, trenches, 126
Tin, atomic weight, 678
corrosion, 808
Tirso dam, data, 141
Titicus dam, data, 166, 184
saturation, 204
Tohickon creek, run-off 69
Toledo, high pressure fire system, 402
rainfall 5
Tombigbee river, run-off, 69
Tompion, wood-pipe 365
Tools, well sinking, 223 233
Topography, ground-water relation, 72
impounding reservoirs, 85
run-off factor 38
Toronto, drifting sand filters, 697
filters 739
high-pressure fire systems, 402
rainfall and temperature, 8
water, bacteria 612
Torrent meters, 464
Tower, intakes, 81
tanks, 563
wooden tanks, 569
Transmission machinery, 520
Transpiration, 36 111
evaporation includes, 26
Transportation of water, Part III, 269 380
Trapezoid, centroid, 133
channel section, 269
Trapezoidal, law pressure, 134
weir, 787
Trees, ornamental planting, 111
Trench, cast-iron pipe, 404, 405
timbering, 126
wood-stave pipe, 361
Trenching machines, 406
Trenton, filters, 741
Triple-expansion, pumping engine 475
Triplex, compound pump, 475
pump, 474
electric drive, 523
Tropics, average temperature, 29
Trout, chlorine kills, 665
Troy, N Y , storage reservoirs, 91
Truck loads, valve chamber covers, 827
Tuberculation, pipe, 322, 387
Tucson, Ariz , rainfall, 5, 21
temperature 21
Tufa cement, 794
Tulip, ornamental planting, 111
Tulsa, conduit, 375
Tumalo reservoir, leakage, 108

Tunnels, aqueduct, 290–296 (see *Aqueducts*)
grade, gradients, 281
ground-water admitted, 291
use, 280
intake, 82, 292, 293
leakage, 294
life, 836
lining, 291–294
pressure, 291–296
depth of cover, 291
gradients, 281
leakage, 294
lining 291
vs pipe lines, 280
use, 280
reservoir outlet, 94
Turbidity of water, alum quantities, 656
coagulation required, 650
coefficient 594
origin, 601
records American rivers 63
removal coagulating basins, 731
rapid filters 729
test 585 594
Turbine, meters 464, 465
pump, centrifugal, 483
wells, 248
steam, 497–501
vs Diesel engine, 528
economy 499
makers, 497
merits, 499
operating cost 521
regulations, 498
stuffing boxes, 499
types, 498
use, 497
Turbo-centrifugal, 497–501 (see *Turbine* and *Centrifugal pumps*)
defined, 471
economy, 499
existing plant 501
performance 477
vs reciprocating pumps 499
types, 499
Turlock dam, data 162 173
Typhoid, bacilli ice 613
longevity, 611
tests 589
vitality 637 638
fever, death rates, 592

U

Ultra-violet rays, 665
Umbrella well head, 258
Underdrains, rapid filters 723
slow filters, 708
Underground water, (see *Ground water*)
Uniflow pumping engine, 475
Uniformity coefficient, 86 703
United States, altitudes tables 826
army cantonments 359 367 423 424
barometer records, 826
cold period 825
Reclamation Service, concrete specifications 796
dams uplift 119
earth rolling specs, 207
evaporation studies 31
ingot iron pipes 338
metal flumes 277
open channels, 269, 270
pipe coatings 319
wood pipe, life, 370
Treasury, water quality standard 590
Weather Bureau, evaporation formula, 30
rain gage, 1
rainfall data, 1, 2, 7–23
wind velocity records, 826
Universal, cast-iron pipe, 395
Unwin formula, pipe flow, 754
Upper Deer Flat dam, statistics 220
Upper Otay dam, data, 167, 184
Uranine, ground-water studies, 72
Urbana, deferrization plant, 701
Urft dam data, 167, 184
Uroglena, odor, 614
Uruguay river, run-off, 63
sediment, 63

V

V-notch meter, 459
Vacuum, conversion to inches, 817
overflow dams, 147
steam boilers, 511
Valuation, materials, life considered, 836
waterworks 830
Valves, air, 446–448
Crispin 447
design 446
discharge 448
flow of air 772
negative-pressure 447
use 446
vaults, 359 447
wood stave pipe 372
altitude 451, 536
butterfly, loss of head 772
chamber covers loads 827
standpipe 551
check, 451, 452
cross connections 403
loss of head 770
substitute 443
dam, replacement shutters 105
distribution reservoirs 536
flap 453
foot 452 489
gate 436 446 (see *Gate valves*)
globe loss of head 772
hydraulic, 453
life 836
miscellaneous forms 451 454
needle 105 453
plug drain 297
pressure regulating, 448 451
Anderson 451
choice 449
location 449
loss of head 772
Ross 449
types 448 450
use 448
quick operating 454
relief water hammer 782
throttling removable, 82
Van Stone joint, 340
Variation, coefficient 89
Vault, air valve 359 447
gate valve 440
Vegetable acids, weak 654
Vegetation, clearing 109
evaporation, 37
floating bottom, 110
rainfall intercepted, 66
run off factor, 39
watertable relation 72 114
Velocity, critical, channels 271
diagram 753
equivalents 816
eroding 106
ground water measurement 74
head diagram 753
high absorption 274
rivers 60
meters 464
particles moved 753
scouring 753
sluicing, 216, 217
streams, variation, 41
subsiding, particles, 640
Venturi, effluent controller 732
meters, chemical feed 668
description 459
filters 716
hydraulics, 460, 461
installing, 461
pump pulsations, 461
reducing bend, 459
tube proportions, 460, 462
Verdigris, 808
Verdite, zeolyte, 687

Vermeule, evaporation studies, 27
Vicksburg, Miss., rainfall, 5
Victaulic joint, pipe, 394
Villar dam, data, 167, 185
Vitrified pipes, 373
flow, 768
Voids, determination, 79
Volvox, odor, 614
Vyrnwy dam, data, 162, 173

W

Wachusett, aqueduct, bridges, 289
cleaning, 297
cracks, 287
section, 282, 283
tunnel, unlined, 296
channel, regulating dams, 276
dam, data, 167, 186
ice thrust, 118
masonry laying, 159
uplift, 116, 167
dikes, saturation, 204
wooden sheeting, 194
north dike, statistics, 219
reservoir, cemeteries, 104
color reduction, 634, 636
forestry, 112
stripping, 109
water analysis, 623
river, run-off, 46, 67, 68
watershed, rainfall and temperature, 10
Wall castings, 382
Walls, core, for dams, 193–198
counterforted, 542
cut-off, dams, 126
reservoir, 541
Walnut Grove dam, failure, 126
Waltham, Mass., rainfall, 3
standpipe, 550, 554, 556
Wanaque, aqueduct, concrete mix, 283
dam, core, 197
flashboards, 103
section, 282
slopes, 281
stream control, 206
watershed water, quality, 609
Warsaw, Poland, rainfall, 23
Washington, D.C., cold periods, 825
filters, 710, 740
rainfall, 5, 14
subsiding basin, 642
temperature, 14
water analysis, 620
Washington fir, wood-stave pipe, 360
Waste, of water, 423–426
weirs, design, 146–148, 161, 162
Water, acidity, test, 598
air in, 779
analysis (see *Examination of water*).
bearing geology, 75
boiling, 583
point, 507
bottled, use, 71
bubbles, rate of rise, 780
character, 583–624
classes, characteristics, 601–624
Collection (Part II), 81–268
color, coagulation, 650
pure, 583
reduction, 634, 635, 655, 729
standard desired, 637
tests, 588, 594
colored, coagulation, 654
purification, 651
companies, charges, 402
compensation, 93
concrete, 796
consumption, (see below).
contamination, sources, 637
control, masonry dam, 157
costs, 832
cushion, overflow dams, 148
Delivery of (Part III), 269–380
density, 583
discharge, equivalents, 819
Distribution of (Part IV), 381–581
Water, drop, equivalents, 815
examination, 584–601, (see *Examination*).
famine scare, 88
filtered, storage, 639
freezing, 421, 583
-gas tar coating, 319
gases, absorbed, 584
dissolved, 583
glass, 661
ground, (see *Ground water*).
hammer, direct pumping, 469
pipes, 781–785
practical results, 782
pressure regulators, 449
reciprocating pumps, 476
wood-stave pipe, 372
hard, 680
limestone regions, 76
hardness, 680
significance, 586
tests, 596, 602
units, conversion, 597, 681
wells, 222
horse-power, equivalents, 819
impounded, sunlight effect, 661
incompressible, 583
-level recorders, 569
light absorbed, 584
measures, equivalents, 818
meteoric, characteristics, 601, 622
gases, 603
modulus of elasticity, 781
odors, iron bacteria, 588
significance, 585
tests, 595
polluted, analysis, 591
power, equivalent, 817
pressure upward, masonry dams, 119, 161–167
properties, 583
pure, pH value, 650
properties, 583
purification (see *Water treatment*).
quality, standards, 590
storage effect, 633–639
quantity, equivalents, 815, 819
ram (pump), 502–505
ram (shock), 781
rates, 832
equivalents, 815
red, cause, 661
river, (see *Water, surface*).
safe, guarantee, 625
sampling, 736
sewage detected, 585
soft, open storage, 639
softening, (see below).
solution, power, 583
station, railroad, intake, 81
storage, 85–93 (see *Reservoirs*).
quality effect, 633–639
time element, 626
value, 625
supply, sources, protection, 625–632
yield, (see *Yield*).
surface, bacteria, 610
characteristics, 606
dissolved gases, 606
industrial wastes, 613
mineral matter, 612
odors, 612
organic matter, 607, 612, 613
organisms, 608
pollution, 606
putrefaction, 610
sewage pollution, 607
sources, protection, 625
tastes, 612
surfaces, evaporation, 33, 34
-table, depth, 72, 222
vegetation relation, 72, 114
tastes, iron bacteria, 588
wood pipe, 371
Transportation, Part III, 269–380
treatment, (see below).
turbidity, significance, 585
test, 594

Water, unaccounted for, mains, 423
pump slip, 482
underground, (see *Ground water*).
vapor, tension, 29
volume, 583
waste, 423, 572
pump slip, 482
weight, 815
equivalents, 618
well, hardness, 246
protection, 237, 630
wholesome, analysis, 591
year, 3
Water consumption, 571–582
agriculture, 575
cantonments, 571
per capita, 571, 573, 576–581
increase, 467
distribution losses, 423–428
domestic, 571
English statistics, 573
fires, 402, 572
forecasts, 575
hotels, 571
iced water, 571
industrial, 571, 579
large cities, variation, 534
lawns, 571
leakage, 425–428
metering, effects, 468, 573
night flow, 575
pressure relation, 400, 426
public uses, 572
pumping engines, condensing, 511
sewer system, 423
slip, pumping engine, 482
sprinkler systems, 403
statistics, 572, 576–582
variation, 573, 574, 582
New York, 534
waste, 423, 572
pump slip, 482
water unaccounted for, 423
water-closets, 571
Water softening, 680–694
boiler feed water, troubles, 681, 682
economics, 681
filters, 691
hard waters defined, 680
hardness, 680
lime, 655
municipal plants, 691
Columbus, O., 692
Defiance, O., 693
types, 688
operating, cost, 738
results, 691–694
processes, 682–694
barium carbonate, 685, 689, 693, 694
boiler compounds, 687
carbonation, 686
caustic soda, 684
economy, 682
heat value, 690, 691
hot, 691, 694
lime-barium, 685, 689, 693, 694
lime-soda, 682, 685, 693
vs. zeolyte, 686
overtreatment, 685
permutit, 686
Reisert, 689, 694
soda ash, 656
Sorge-Cochrane, 691, 694
types, 687
We-fu-go, 689
zeolyte, 686, 687
soap, 681
sulfate limits, 682
weighing chemicals, 673
Water treatment, (Part V), 645–745
aeration, 644–649
coagulation (see *Coagulation*).
colloidal clay, 633
contact baffles, 695
corrosion effect, 661, 810
costs, 738, 832
deferrization, 699–701
Water treatment, degasification, 695
demanganization, 701
disinfection, 661–666
distillation, 695
drifting sand filters, 697
filters, double, 696
intermittent, 696
preliminary, 695
rapid, 716–731
slow, 705–716
filtration, 702–745
meters affected, 465
pipe, cleaning effect, 423
incrustations, 813
rain water, 695
scrubbers, 695
sedimentation 633–639, 640–644
statistics, 703, 739 745
in tropics, 697
Waterbury dam, leakage, 194
Watering station, public, 430
Waterproofing concrete, 799 801
tanks, 550, 554
incorporated, 801
Watershed, defined, 38
development by reservoirs, 85–115
leakage, 108
overflow prevented, 92
reforestation, 110
run-off, (see *Run-off*).
formulas, 61
sanitary patrol, 625
sewage disposal, 627
shapes, run-off, 39
slope, evaporation factor, 27
small, minimum run-off, 64
yield, diagram, 68
studies, 87
Watertown, subsiding basin, 642
Waterwheels, pump drive, 525
Waterworks, appraisals, 829
bonds, 832
costs, 832
depreciation, 830, 832
financing, 829–836
labor, cost, 831
materials, cost, 831
ownership, public vs private, 829
state regulation, 829
valuation, 830
Waves, grassing protection, 112
height, 199
paving injured, 201
Wedge-rod method, wells, 225
Weight, metric, conversion, 584
Weirs, flow measured, 785 791
Bazin's formula, 785, 791
box, 790
discharge, diagram, 97
tables, 786
error, 42
Francis formula, 785
proportional, 787
rectangular, 785–787
Sutro, 788, 791
trapezoidal, 787
triangular, 787, 790
overflow, 93–104
aqueducts, 281
on pervious foundations, 149 157
skimming, 536
waste, design, 146–148, 161, 162
economic length, 97
Weld, strength, 347
Welded pipe, 348–355, (see *Steel pipe*).
well casings, 224
Welders, selection, 346
Welding, autogenous, 348
cast-iron pipe, 420
forge, 348
fusion, 348
hammer, 348
steel pipe, 346, 347
Well, 222–259
areas, flooding, 74
arrangement, 238
artesian, 75

Well, artesian, jetting, 228
back-blowing, 231, 244
bacteria enter, 628
casing, 224, 226, 233
corrosion, 246
dimensions, 351
safe, 630
sinking, 226, 230
clogging, small diam., 75
combination dug and drilled, 235
concrete, 234
covers, 235, 631
safe, 630
curbs, 233, 631
deep, dryness, 73
protection, 630
purity, 604
vs shallow, 222
sinking, 223
depth measurement, 242
drainage, masonry dams, 119, 160
drill hole deflection, 237
drilling methods, 223–233
prospecting 72
drive pipe, 224
driven, objections, 233
dug, 233
in clay 76
vs drilled, 222
pollution, 628
protection, 631
earthquakes, 828
farm yard, bact coli, 604
flow, 238, 241
gravel packing 238
hydraulics, 238
vs infiltration galleries, 260
interference, 239, 245
kinds, 222
large vs small, 222
leak detection, 246
life 243
lost supply, 243
maintenance 243
percolation, 231
polluted odors, 602
pollution, 222, 606
protection 233
pumping rate 243
pumps 247–259
air-lift, 249–259 (see *Air-lift*)
centrifugal 248
efficiency 248
electric, 248
houses 249
manufacturers, 247
plunger 247
power 247
turbine 248
in rock, futile 73
salt-water interference, 245
sand excluded, 237
screen, 237
damage, 244
drive point, 231
screening, rotary method, 228
shallow, vs deep, 222
protection, 631
purity, 604
sinking, 231
shooting, 230
sinking methods, 223–233
auger boring, 231
boulders, 235
British practice, 230
California method, 223, 225
chilled shot, 230
clays, 235
core drills, 229
difficulties, 235
driving pipe, 232
dug wells, 233
explosives, 230
hollow-rod method, 225
hydraulic rotary method, 223
jetting method, 223, 228
mud, 235
Well, sinking methods, pole-tool method, 225
precautions, 226, 230
punching, 232
quicksand 236
rotary method, 227
self-cleaning method, 225
spudding, 224
standard method, 223
stovepipe method, 223, 225
tools lost 235
wedge rod method, 225
size, 238
small vs large, 222
spacing, 238
specific capacity, 240
strainers, 237, 244
successful, conditions, 222
suction, 530
supply increased, 230
testing, 240
tools, 223–233
types, comparisons, 222
water, analysis 591, 622
bact coli 604
yield, 74, 75, 240
Wellesley Arch, bridge 290
Westerly, standpipe, 550, 551, 552, 553, 554
Western Australia, rainfall 21
Weston, aqueduct, cleaning, 297
section 282
reservoir core 197
Wet connections, pipes 419
Wiesbaden, subterranean galleries, 264
Wigwam dam, data, 167
Wilkes Barre, storage reservoirs 91
Williams and Hazen formula, 754
Willow, ornamental planting, 111
Wilmington, Del, filters 740, 742
Wilmington, N C, rainfall, 5
Wind, factor, evaporation, 28
load tanks, 558
pressure masonry dams 119
run-off factor 39
velocity records 826
Windmills, 505
vs hydraulic ram 502
Winnipeg, aqueduct burlap coating 318
capacity 280
concrete failure 375
construction 283
deterioration, 797
joints 287
settlement, 282, 286
slopes 281
temperatures 288
high pressure fire system, 402
Wood (see also *Timber*)
chemicals attack 669
decay, 369
for wood stave pipe, 360
Wooden flumes, 276
Wood-stave pipe, 358–372
air valves, 359, 446
bridges, 372
capacity, 358, 766
age effect, 766
coating 371
collapse, 359
conduit details, 372
continuous-stave 360–363
construction, 360
cost, relative, 359
defined, 358
design, 362
laying, 361
vs machine made 365
shoes, 361
staves, 360
tongues, 360
cost 372
relative, 358, 359, 369
creosoted 369, 371
curves, 361
design, 371
disadvantages, 359
distortion, 359
durability, 358, 363, 369

Wood-stave pipe, electrolysis, 360
  essentials, 358
  factor of safety, 359
  factory-made, 358, 363–368
  fire risk, 358
  flow, 358, 766
  freezing, 358
  friction-head table, 755
  Kutter's formula, 766
  leakage, 358, 359, 368
  life, 370, 836
  machine-banded, 358, 363–368
    anchorage, 366
    bands, spacing, 364
    cantonment troubles, 359
    c. i. fittings, 367
    vs. continuous, 365
    cost, relative, 359, 368, 369
    curves, 366
    defined, 358
    design, 366
    Eastern vs. Western, 364
    elbow, 367
    in distribution systems, 367
    joints, 364
    kinds, 363
    laying, 365
    leakage, 368
    manufacturing, 363
    repairs, 366
    service connections, 366, 367
    troubles, 359
    wires, spacing, 364
  merits, 358
  pressures, 358, 359
  principle, 358
  pumping, leaks, 359
  salt marshes, 358, 418
  specifications, 372
  staves, requirements, 358
  vs. steel, 359
  water hammer, 372
  woods, 360
Worcester, storage reservoirs, 91
Wrought, pipe, 349
  steel, 349
Wrought iron, corrosion, 811
  durability, 334
  pipe, commercial, 349
    dimensions and weights, 350
    riveted, 334–338
    vs. steel, 335, 349
    stream crossings, 416
    superiority, 349
    welding, 346
  vs. steel, 335

Y

Y-branch, standard flanged, 345
Yankton, rainfall, 5
Yarn, weight per joint, 412
Yarrow dam, statistics, 220
Year, climatic or seasonal, 3
  rainfall and run-off records, 2, 3
  water, 3
Yield, equivalents, 815, 816
  ground water, 74, 75
  watershed, 87–93
    defined, 38
    storage relation, 85
    water surface effect, 91
  wells, 240
Yuma, rainfall and temperature, 21
  reservoir, silt, 108

Z

Zeolyte, water softening, 686, 687
Zifta regulator, 152, 153
Zinc, atomic weight, 678
  chloride, disinfectant, 632
  corrosion, 808
  poisoning, 587
Zola dam, data, 139, 167, 185
Zuni, dam, statistics, 220
  reservoir, silt, 106